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ACI
PROCEEDINGS
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Contributions to the

ACI JOURNAL

Technical pages of the ACI JOURNAL offer a medium for reporting what is new and interesting in concrete research, design, construction, maintenance, or manufacturing. ACI members and JOURNAL readers are urged to share their knowledge and experience by submitting papers and reports for publication in the ACI JOURNAL. Time and changing interests demand that new contributors with new ideas take their place beside the old.

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AUGUST 1958

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Coming next month in the JOURNAL

"Proposed Recommended Practice for Selecting Proportions for Structural Lightweight Concrete," prepared by Committee 613's Subcommittee on Lightweight Concrete for the September JOURNAL will be considered for adoption as an ACI standard at the 55th annual convention in Los Angeles next February. Subcommittee VIII of Committee 325 reports on "Design of Concrete Overlays for Pavements," also scheduled for the September issue.

●

B. Bresler and K. S. Pister's joint paper, "Strength of Concrete Under Combined Stresses," represents their final report on a project of the Reinforced Concrete Research Council at the University of California.

In "Economic Factors in Prestressed Lift-Slab Construction," Edward K. Rice discusses conventional versus lift-slab construction, pointing out the factors affecting economy.

●

"Origin, Evolution, and Effects of the Air Void System in Concrete. Part 3—Influence of Water-Cement Ratio and Compaction," presents work by J. E. Backstrom, R. W. Burrows, R. C. Mielenz, and V. E. Wolkodoff.

●

Results of investigations by Harold Samelson and Abba Tor to evaluate stresses in walls of underground reinforced concrete tanks are presented in their joint paper, "Evaluation of Stresses in Reinforced Concrete Sections Subject to Transient Temperature Gradients."

"Shear Strength of Lightweight Reinforced Concrete Beams" by J. A. Hanson, describes a study of resistance of lightweight structural quality concrete in diagonal tension.

●

Discussion of papers published in the January, February, and March, 1958, issues will appear in the September JOURNAL, in accordance with the quarterly publication schedule for discussion.

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Curing Concrete*

Reported by ACI Committee 612

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SYNOPSIS

Five fundamental requirements for proper curing of concrete discussed in some detail are: (1) preservation of adequate water content in concrete; (2) maintenance of concrete at some fairly constant temperature above freezing; (3) preservation of reasonably uniform temperature throughout the whole body of concrete; (4) protection from damaging mechanical disturbances; and (5) passage of sufficient time for hydration of cement and hardening of concrete. Specific initial and final curing procedures and materials are recommended for horizontal structures, vertical structures, mass concrete, and precast units.

INTRODUCTION

In this report, optimum curing is defined as the act of maintaining controlled conditions for freshly placed concrete for some definite period following the placing or finishing operations to assure the proper hydration of the cement and the proper hardening of the concrete.

This definition is intended to apply to concrete made with portland cement or blends of portland cement with pozzolanic or other materials and is based on the conditions of moisture, temperature, freedom from physical disturbance, and passage of time that have been found to be essential to its proper hardening. If, within the curing period, natural temperatures of the concrete are within the acceptable range of values, only the moisture content needs to be controlled. If the natural temperature will be outside of the acceptable range of values, some means for the control of the temperature of the concrete will be required. Control of temperature throughout the curing period is desirable, but except for certain structures, it is less essential than other requirements. Uniformity of temperature throughout the mass of the concrete is desirable and in certain structures essential for the production of a satisfactory result. The concrete mass should be kept free from damaging mechanical disturbance during the

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This report was submitted to letter ballot of the committee which consists of ten members, with nine voting affirmatively, and one negatively.

curing period. Load stresses, heavy shocks, and readily apparent vibrations especially should be avoided.[5] Finally, time must be allowed for the hydration of the cement and hardening of the concrete.[2,3] The amount of time varies with the rate of hydration and the degree of hardening needed for safe use. For hydraulic cements, these several conditions are the major factors other than the characteristics of the individual cement pertinent to the curing process.

BASIC CURING REQUIREMENTS

There appear to be five fundamental requirements for the proper curing of concrete.[4,6,7,8,10,16] Each of these may be modified by the volume, mass distribution, and degree of exposure of the concrete and extent of the work in which it is used. The five requirements are:

1. The preservation of an adequate water content in the concrete.
2. The maintenence of the temperature of the concrete at some value above freezing as constantly as may be practicable during the curing period.
3. The preservation of a reasonably uniform temperature throughout the whole body of the concrete.
4. The protection of the structure from damaging mechanical disturbance, particularly load stresses, heavy shock, and excessive vibration especially during the early portion of the curing period.
5. The passage of time for the hydration of the cement and hardening of the concrete to the degree necessary for the safe use of the article or structure which it forms.

Preservation of moisture content

The first requirement for the proper curing of concrete is the preservation of an adequate moisture content in the concrete following the completion of the placing operations. Water may be taken advantageously from the unhardened concrete if this is accomplished without removal of cement and is accompanied by a corresponding consolidation in the mass of concrete.

Preservation of the moisture content of the concrete may be accomplished by any of several methods or by a combination of two or more methods. The surface of the concrete may be kept wet with water, or loss of moisture may be prevented or restricted by the application of impervious coatings, membranes, or coverings, or by the retention of forms. Forms alone without curing water added are an inadequate curing medium in warm and arid regions and they usually leave some surfaces exposed. Changes in both temperature and moisture content of the concrete may be minimized through the use of some one of a variety of coverings, blankets, or other media. Specific agencies employed in these several methods will be described in detail in the sections of this report dealing with the curing of concrete in the various types of structures in which it may be employed.

Control of temperature of concrete

Curing of concrete may be accomplished in a wide range of temperatures. The rate at which hydration of the cement occurs varies with the temperature of the concrete, proceeding slowly at low temperatures somewhat above the freezing point of water, and rapidly at high temperatures somewhat below the boiling point of water. Concrete will cure at any low temperature above freezing; it is necessary only that sufficient time and moisture be provided for it to obtain required strength, and that it be protected from freezing. There is some evidence[22] that curing at temperatures in excess of 150 F impairs the ultimate quality of concrete and therefore should be undertaken only under expert supervision and under carefully and positively controlled conditions.[7] Some block plants steam cure at 180 F to provide high early strength without significant harm to the block for usual construction. Actually, little benefit to early strength is realized by curing at 180 F instead of curing at 150 F,[22] except possibly where fly ash is used in the mix.[23]

Methods employed for insuring a temperature of concrete within the range of acceptable values depend upon surrounding atmospheric conditions, length of curing period, type of structure in which used, extent of the work in which it is used, and volume of concrete involved. For protection against low temperatures, heating of the mixing water, heating of the mineral aggregates, and protecting the concrete after placement either by means of heated or unheated shelters or by use of coverings of insulating materials are required in varying degrees of thoroughness depending upon the atmospheric temperature, the extent of the work, and the volume and distribution of the concrete.[9] Protection against high temperatures is a somewhat more complex procedure as the excessive temperature of the concrete may be derived from any one or all of three sources: the temperature of the surrounding atmosphere, absorption of solar heat, and the heat of hydration of the cement.[7] As the temperature of the surrounding atmosphere is unlikely to exceed the maximum value for the range of acceptable temperatures except in curing chambers used for manufactured precast products, the principal needs are for protection against absorption of solar heat and excessive heat of hydration of the cement. Reflective or insulating materials should be used for protection from the sun when concrete is exposed to its rays during the curing period. Excessive temperature rise due to heat evolved by the hydration of the cement may be avoided by the use of cements or cement blends of low heat of hydration, reduced cement, precooled materials, or, in certain cases, the use of cooling systems to remove heat as it is liberated.

Logically, the ideal curing temperature for a given concrete installation or structure may be a few degrees below the average temperature to which the concrete will be exposed during its period of service. Under this assumption, concrete placed and cured when temperatures are below this average has two requirements which should be met: (1) the temperature of the concrete when placed should be above freezing, and (2) during the curing period the concrete

should be kept between freezing and the average annual temperature of the atmosphere, preferably nearer the latter. Furthermore, at the end of the curing period, precautions should be taken to allow the concrete throughout its whole mass to approach gradually the temperature to which it is to be immediately exposed. The drop in temperature during the first 24 hr after curing ceases should be no more than 30 F for mass concrete or 50 F for thin sections.[7,24]

The major portion of the concrete construction of any region is probably undertaken when air temperatures are near or above the average annual value. The problem of curing in these circumstances involves the prevention of the loss of moisture from the concrete and the protection of the concrete against temperatures greatly in excess of the average annual value due to temperature of the air, the heat of hydration of the cement, and the absorption of solar heat.[5,13,14,15,16,20] In most cases it is impractical to maintain a temperature of concrete much if any lower than the temperature of the surrounding atmosphere. It is practicable and highly desirable to avoid temperatures in the concrete appreciably above that of the surrounding atmosphere. This may be done through appropriate curing methods which insure at least that much control of the temperature of the concrete as well as control of the moisture losses from the concrete.

In mass concrete construction, the aggregates may be cooled artificially, cement with a lower heat of hydration may be used, and, if early low internal temperatures are desired, a pipe cooling system may be installed within the concrete, all for the purpose of reducing and controlling the temperature rise of the concrete. In such circumstances it is desirable to place the concrete at as low a temperature as practicable and to maintain a relatively uniform temperature throughout the concrete during the curing period.[21] In those circumstances curing is largely confined to this control of the temperature of the concrete.

Temperature control for manufactured precast concrete may be in general more readily accomplished than for other concrete work. For most efficient use of the relatively smaller working space for this type of concrete work than is used for the other types it is desirable to reduce the curing time to a minimum in order that the finished product may occupy the curing space for as short a time as possible. Curing temperatures, usually provided by the use of steam, may be high to accelerate curing but should not exceed 180 F, preferably 150 F, at atmospheric pressure and should not fluctuate rapidly enough to cause damage to the product.[12]

Physical disturbance of concrete

Early progress of the hydration of the cement in the concrete is indicated by the setting or solidifying and finally the hardening of the mass. Continuous mechanical disturbance of the concrete during this process will result in the failure to form the integral solid intended in the design of the structure,

and the member or structure will fail to attain the strength for which it was designed. During the setting period and for a short time thereafter concrete has little ability to take stress. The ability to take that for which the member or structure was designed is attained in the curing process which provides a favorable opportunity for the adequate and early hydration of the cement. Premature stressing or loading of the concrete is therefore to be avoided. Damaging shock from explosives, from heavy falling objects, or from other construction operations should also be avoided.[8]

Length of the curing period

The amount of time required for the concrete to attain the strength desired for safe use or to attain the desired degree of hardness varies with the temperature at which the concrete is cured, the rate of hydration of the cement used, and the availability of moisture for hydration of the cement. When the moisture content of the concrete is maintained at or near that at the time of placing, the length of the curing period depends upon the temperature of the concrete and the rate of hydration of the cement used. For freshly placed concrete, hydration of the cement proceeds slowly at temperatures near the lower limit of the acceptable range and rapidly near the upper limit of that range.

In favorable circumstances, hydration of the cement appears to continue for a long period at a decreasing rate after the concrete is placed. As a consequence, hydration subsequent to any practicable length of curing period is at a much lower rate than for the early hours and days immediately after the placing of the concrete. Considerable evidence is available that hydration of the cement may cease at any time because of lack of available moisture or because of unfavorable temperatures in the concrete and will resume when moisture is again available and temperatures are favorable.[4,6] The improbability of continuous favorable circumstances and the low rate of hydration of the cement in hardened concrete combine to establish the virtual necessity of obtaining the greatest possible percentage of useful hydration of the cement under the controlled conditions provided in an appropriate and adequate early curing period.

Acceleration of hydration of cement

Considerable effort has been expended in studies of methods for shortening the curing period. As a consequence it has been determined that provision may be made in some work for shortening the curing period through the use of accelerators in the concrete, through the use of cements having a high rate of hydration, or through elevation of the temperature to a value near the upper extreme of the acceptable range of temperatures. These special procedures for the reduction of the length of the curing period should be used under the direct supervision of personnel skilled in their use in the type of construction being undertaken.

RECOMMENDATIONS FOR OPTIMUM CURING

Classifications of structures

1. Horizontal units are those in which the principal surface of the concrete is in a horizontal position and large in proportion to the vertical surfaces and in which the least dimension of concrete is 2 ft or less.

2. Vertical units are those in which the principal surfaces of the concrete are in a vertical position and large in proportion to the horizontal surfaces and in which the least dimension is 2 ft or less.

3. Cubical units are those in which the least dimension of concrete is more than 2 ft, either for that placed in a single operation or that placed in a series of casting operations to form a structure having a least dimension in excess of 2 ft.

4. Precast units are those in which the concrete product is cast and finished in a place or position other than that which it is to occupy in service.

Typical examples of the types of units classified as horizontal units are pavements, sidewalks, canal linings, roofs, and small docks; typical of those classified as vertical units are walls, small piers, small abutments, small dams, and small columns; typical of those classified as cubical units are large piers, large abutments, large dams, and other massive concrete structures in which the least dimension is more than 2 ft; and, typical of those classified as precast units are concrete pipe, building block, brick, precast structural members, burial vaults, piles, and fence posts.

Optimum practice for curing horizontal units: pavements, sidewalks, canal linings, small docks, and building floors

Initial curing—Immediately after the finishing operations are completed, the concrete should be covered with two thicknesses of an approved woven fabric, a quilted fiber mat, or other covering of an approved absorptive material thoroughly saturated with water when placed. This covering should be kept saturated with water by spraying until removed. This covering should remain in place at least until the peak of the temperature of the concrete produced by the heat of hydration of the cement has been passed. This period will vary with the initial temperature of the concrete, the temperature of the surrounding atmosphere, and rate of hydration of the cement used, but should be at least overnight.

The initial curing may be discontinued at the end of the initial curing period provided that an acceptable substitute curing agency be used for the remainder of the curing period. Curing agencies and procedures which may be substituted in the final curing period for the wet coverings used in the initial curing period should be applied immediately after the removal of those coverings, and before the surface of the concrete has had an opportunity to become dry.

Final curing—An acceptable curing agency for application after the initial curing period and for the final curing period may be chosen from the following list.

1. The covering used in the initial curing period kept in place and in the same condition as for the initial curing period.

2. Two in. of moist earth or sand evenly distributed over the surface of the concrete and kept thoroughly saturated by spraying with water.

3. Three in. of moist cured hay, grass, or clean straw uniformly distributed over the concrete and kept wet by a water spray.

4. Approved impervious light colored paper or plastic coverings placed and maintained in contact with the surface of the concrete.

5. Approved impervious coatings which may be developed from liquid compounds sprayed on the surface of the concrete. When concrete is exposed to the rays of the sun these coatings should be light colored. If darker in color than the concrete, they should be covered completely with a heavy coating of whitewash as soon as they have hardened. This is necessary to avoid absorption of solar heat.

Alternative combined initial and final curing—In certain circumstances, initial curing under water-saturated covering may be impractical or impossible in a given location or for a given series of projects. For example, (1) materials for use in woven fabric or quilted coverings may be unavailable, (2) temporary governmental restrictions on the use of these materials for curing concrete may prevent their usage for curing, or (3) water spraying on completed work may interfere with construction operations on adjacent new work.

In such circumstances, the best alternative is the application of approved impervious light colored paper or approved impervious coating materials directly upon the surface of the wet concrete either immediately following the completion of the finishing operation, or, at the latest, immediately following the disappearance of the water sheen from the surface of the concrete. This practice will usually permit somewhat greater moisture loss during the early curing period than that which would be attained were the concrete cured initially under wet coverings and finally under impervious paper or impervious coatings.

Final decision in the selection of the method of curing must balance the disadvantage of this slight deficiency in strength against the advantages of the services which may be made available only through the construction of concrete of somewhat lower quality than might be had under more favorable circumstances.

Temperature—When the temperature of the atmosphere is 40 F or more, the final curing agencies should remain in place for at least 72 hr and for such additional length of time as may be required for job cured specimens to have attained the compressive or flexural strength required of the concrete at the age tested. When the mean daily temperature of the atmosphere is less than 40 F, the concrete should be so protected as to maintain within it a temperature of 50 to 70 F for the curing period required.

Accelerators—Approved accelerating agents may be used with normal setting cements, or cements with a rapid rate of hydration may be used to reduce the length of the curing period in the event urgent need for the facilities being provided is indicated. Field specimens should be prepared and tested to determine the length of curing period required for the concrete to attain the desired

hardness and strength in each case where an accelerating agent or cement with rapid rate of hydration is used.

Other surfaces—Edges of pavements, sidewalks, or other structures supported by forms during the placing and finishing operations should be sealed with an impervious coating material immediately after the removal of the forms in the event the particular curing method in use on the job fails to provide proper curing of these surfaces.

Optimum curing for vertical units: walls, small piers, columns, floors, roofs, small dams, and small abutments wherein the least dimension is less than 2 ft

Initial curing—Immediately after the finishing operations are completed or upon cessation of placing operations for a period of more than 3 hr, the exposed surface of the concrete should be covered with two thicknesses of an approved woven fabric, an approved quilted fiber blanket, or other cover of highly absorptive material thoroughly wet with water when applied. This covering should be kept saturated with water by spraying with water until removed. This covering should remain in place for at least 96 hr. Except where required to support concrete subject to dead-load flexural stresses or the forms supporting concrete subject to such stresses, the forms in which the concrete is placed may be removed after 24 hr and any of the methods used for initial curing of exposed surfaces applied for the balance of the 96-hr curing period.

If for any reason it becomes necessary or desirable after the expiration of 96 hr to discontinue the initial curing practices, or to remove the forms before the concrete has attained the strength desired, provision should be made for additional curing under controlled conditions and for continued support of the concrete wherever dead-load flexural stresses in the concrete develop upon removal of the forms.

Final curing—Upon removal of the wet coverings used for the initial curing, any one of the following methods for final curing practical or desirable in a given project may be used.

1. Application of a continuous mist spray of water directly on the concrete.
2. Application of an approved impervious paper or plastic covering directly upon the surface of the concrete.
3. Application of an approved impervious coating material upon the concrete.
4. Application of a layer of wet sand at least 2 in. thick (top of horizontal portions only).

Temperature—When the temperature of the surrounding air is 40 F or more the total length of curing period should be at least 10 days, the initial curing period at least 4 days, and a final curing for the remainder. For temperatures near 40 F the final curing period should be continued beyond 10 days for such additional time as may be required for job cured specimens to have attained the compressive or flexural strength required of the concrete. When the mean daily temperature of the surrounding air is less than 40 F the concrete

should be so protected as to maintain within it a temperature of at least 50 F to 70 F for the curing period.

Accelerators—Accelerating agents may be used with normal setting cements or cements with a rapid rate of hydration may be used to shorten the curing period. Amount of reduction in length of curing period should be determined from strength tests on field specimens of concrete used in the structure and subjected as nearly as may be possible to the same curing conditions as the concrete in the structure.

Transition period—Care should be taken either at the end of the curing period or upon removal of the forms to avoid sudden exposure to temperatures greatly different than that at which the concrete was cured. Particular attention should be given this matter in cold weather when exposure should be made in such manner as to allow time for the whole mass of concrete to approach gradually the temperature to which it is to be subjected immediately following the curing period. In no case should the temperature change at the suface of the concrete be greater than 5 F in any hour or be greater than 50 F in 24 hr.

Optimum curing for cubical units: large piers, large abutments, large dams, and other massive concrete sections in which the least dimension is 2 ft or more

1. As a prerequisite to proper temperature control of mass concrete, arrangements should be made for the temperature of concrete as placed to be as low as practicable, but consistent with economy and design considerations. In any event, the internal concrete temperature during hydration should not reach a value in excess of 20 F above the mean annual temperature. In work of sufficient importance and magnitude to justify the expense, arrangements should be made when necessary for the installation of a cooling system within the concrete mass for the reduction of the temperature of the concrete to the extent required to maintain a relatively uniform temperature through the whole mass of the concrete and to keep temperature of the mass within the range for proper curing. In a similar situation, the use of a cement with a low heat of hydration is advisable. Instead of by a cooling system within the concrete mass, the concrete can be artificially cooled before placing by adding ice at the mixers, or by inundating the coarse aggregate in ice water, or by passing cold air through the aggregate bins, or by a combination of these methods. In extremely large structures, any of these methods can be used in combination with a cooling system within the concrete mass.

2. Immediately following the cessation of placing operations for any reason, action should be taken to keep the exposed surface of the concrete continuously wet.

3. Immediately following the removal of forms, the newly exposed surfaces should be kept wet when air temperatures are in excess of 40 F. Forms may be removed as soon as the concrete has hardened sufficiently to stand without lateral support under weight of superposed concrete and forms for that concrete. In mean daily ambient temperatures less than 40 F the exposed concrete should be sheltered and maintained at some temperature above 40 F.

4. For outside surfaces of the concrete mass the wetting should be continued for at least 3 weeks for concrete made with low heat cements and for 2 weeks for concrete made with normal or modified cements.

5. For the vertical construction joints between sections and for the horizontal construction joints introduced through the suspension of placing operations, the wetting should be continued until just prior to the preparation of these surfaces for the resumption of the placing operations which will cause them to be covered with fresh concrete.

6. Immediately following the removal of forms, liquid membrane-forming compounds may be applied to the newly exposed surface in lieu of the method of treatment prescribed in Paragraph 3; providing these surfaces are not to be covered subsequently with concrete or the appearance of the exposed surfaces in the finished structure will not be adversely affected by the application of such compounds.

7. In the event a long continued period intervenes between placing operations the vertical joints should be kept wet for the specified curing period in the manner prescribed for outside surfaces in Paragraph 4 above. If the vertical joints are to be exposed for periods of more than 30 days, the concrete may be protected, after removal of the forms, with a nonabsorptive insulating board until these surfaces have been covered with concrete from the adjoining blocks. Horizontal joints should be cured by placing and maintaining upon the concrete for a period of 3 weeks, a 2-in. layer of wet sand.

8. In the preparation for the resumption of casting operations, the surfaces to be covered by fresh concrete should be kept wet for 3 days immediately preceding these operations.

9. Care should be taken either at the end of the curing period or upon removal of the forms to avoid sudden exposure to temperatures greatly different than those maintained for the curing period. Particular attention should be given to this feature in cold weather. Insofar as possible, exposure should be made in such manner as to allow the whole mass of concrete to approach gradually the temperature to which it is to be subjected immediately following the curing period.

10. Accelerators to increase rate of hydration of cement and cements with a rapid rate of hydration should be avoided in mass concrete because they increase the rate of heat evolution.

Optimum curing for precast units: concrete pipe, building block, precast structural members, fence posts, burial vaults, and garden furniture

Initial curing—Immediately after the completion of the casting operation for molded precast units, each article should be covered or enclosed by two layers of an approved water-saturated fabric until placed in position for final curing. The length of initial curing for units going into final steam curing will vary with steam curing temperature. From 1 to 4 hr is indicated, the higher the temperature, the longer the period.[22]

Final curing—For final curing each article may be cured in the place in which cast under the original covering which must be kept thoroughly saturated for the entire curing period. For final curing each article may be moved at any time to a special curing chamber, where it may be left uncovered in an atmosphere completely saturated with either a mist spray of water or of steam. In lieu of this treatment, final curing may be accomplished under two layers of an approved wet fabric thoroughly and continuously saturated with water for the entire curing period. The temperature of a curing room at atmospheric pressure should be maintained uniformly at some value between 50 and 180 F. Final curing may be performed under a pressure between 100 psi and 150 psi in saturated steam at 335 to 366 F.

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APPENDIX—APPROVAL OF MATERIALS

At intervals throughout this report reference to "approved materials," or "approved," in the designation of a material to be used in curing concrete implies reference to a standard specification or standard methods of testing for the quality of material as a prerequisite to the use of a given material for curing concrete. In this report it is therefore assumed that the basis of approval of such materials will be the generally accepted and authoritative standard specifications for quality, for composition, and for fabrication, and the results of the generally accepted and authoritative standard methods for testing these materials. These generally accepted standards are those of the American Society for Testing Materials, American Association of State Highway Officials, American Standards Association, and agencies of the federal government.

Discussion of this report should reach ACI headquarters in triplicate by Nov. 1, 1958, for publication in the March 1959 JOURNAL.

Pressures on Formwork*

Reported by ACI Committee 622

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SYNOPSIS

This report is a preliminary step in the preparation of a recommended practice for design and construction of concrete formwork. It presents recommended design assumptions for lateral pressure of concrete on vertical formwork when revibration and external vibration are not to be used. It is intended to provide a safe guide within the range of most formwork and common construction conditions.

Part 1 was compiled from data first available, and on that basis certain recommendations were made (p. 178). Subsequently more data became available and are discussed in Part 2. From these further studies, modifications were derived in the conclusions (p. 189).

PART 1—DISCUSSION OF VARIABLES AND DEVELOPMENT OF FORMULA

History

Subcommittee I of Committee 622, consisting of William R. Waugh, chairman, H. P. Cerutti, David E. Fleming, Donald Peirce, P. R. Stratton, and William H. Wolf, was organized to review the test reports and design formulas in use, and to formulate a safe design formula for lateral pressure on forms. The review covered material published from the early 1900's to the present. One fact is apparent; namely, the lack of agreement. Laboratory investigators, mathematicians, form-tie manufacturers, and contractors still disagree on variables and conclusions after 50 to 60 years of discussion on this subject. Fifty-seven percent of the 75 contractors responding to a nation-wide query of a selected cross section by Subcommittee II on this subject responded that

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This report in form and substance as here submitted was approved unanimously by the committee as listed above.

they used a fluid pressure of 150 psf per ft of depth in their form design.* This can be either wasteful or hazardous, depending on the rate of placement and temperature of concrete.

The early investigators who pioneered in this work were faced with the difficult problem of determining what the significant variables were and then evaluating their effect on the lateral pressure of the concrete. From the work of each investigator one can get a general idea of the variation in pressure due to a change in the quantities with which he dealt, such as rate of placement, slump, etc. However, there is little correlation, if any, between the quantitative results of the different investigators due to the lack of uniformity in the method of testing. Where one individual might have determined the form pressure using the height of placement and slump as the variables, another might have disregarded slump and height of placement but considered rate of placement and temperature only. In addition the early results varied widely due to a lack of uniformity in batching, mixing, and particularly placing of the concrete.

Some of these differences have passed with the years as control of concrete construction has become well defined and accepted. Some of the variables have also become minor in nature due to the widespread use of mechanical vibration. For instance, whereas slump was considered an important factor by early investigators, mechanical vibration of today has minimized its importance as a variable. The same can be said for aggregate size, water-cement ratio, form friction, and cohesion. This is due to the agitation and resulting liquefaction of the freshly placed concrete with the modern immersion vibrator, operating at speeds of not less than 7000 rpm for vibrating heads less than 4 in. in diameter and not less than 6000 rpm for vibrating heads 4 in. or more in diameter.

However, vibration, revibration, and external vibration cause new problems because of their effect on the fluid concrete, and they unquestionably are variables of major importance. It will thus be found that much of the confusion between early investigations, foreign practice, and contemporary standard practice centers around the interpretation or omission of this factor. Moreover, the importance of some variables, such as temperature, has become evident as the conditions have become better defined.

Today there is an encouraging similarity between investigations and the methods used by leading form manufacturers in the United States. In fact, after considering the competitive industry in which the latter operate and the variety of their source material and investigations, the committee finally decided that a comparison of the manufacturers' results offered the most promise of an answer. Happily, the committee found correlation among the policies and assumptions which it is possible to confirm, and the committee recommendations follow along the lines of many of the manufacturers' basic conclusions.

*Subcommittee II, ACI Committee 622, "Form Construction Practices," ACI JOURNAL, June 1957, *Proc.* V. 53, pp. 1105-1118.

Variables

The following variables have been measured at various times and have been found to affect the lateral pressure of concrete after placement in forms:

1. Rate of placement
2. Consistency of concrete
3. Weight of concrete
4. Maximum aggregate size
5. Temperature of concrete mix
6. Ambient temperature
7. Smoothness and permeability of forms
8. Cross section of forms
9. Effect of consolidation by vibration
10. Placing procedures
11. Pore water pressure
12. Type of cement
13. Depth of placement

Rate of placement, temperature of concrete mix, and effect of consolidation by vibration are the major variables in most cases. A discussion of all the variables follows.

1. Rate of placement (in ft per hr)—Rate of placement is assumed to be a uniform rate of rise of the concrete in the form. This is recognized by all investigators and contractors as having a primary effect on lateral pressures regardless of the over-all height of the lift and is directly proportional to the pressure.

2. Consistency of the concrete—An approximate idea of the consistency of the concrete can be obtained from the slump test. This consistency can be expressed as a function of cohesion and internal friction and varies with time as set occurs. The fact that this internal strength is appreciable at time of placement is evidenced by the fact that very dry and properly vibrated concrete will stand to a vertical height of 3 to 4 ft if the forms are removed immediately after vibrating. Vacuum treated concrete will stand in the same way to a height of 15 ft, according to Schjödt.[1] However, variations in consistency are disregarded by most investigators and form-tie manufacturers, and the committee feels that they may be disregarded. Although the committee recognizes that the use of high slump concrete is fairly common today, for good practice the trend is to use slumps of 4 in. or less. In this way the variation in consistency is considerably narrowed. Also, since the mix proportioning will vary only slightly from job to job, assuming small variations in the water-cement ratio and ultimate strength requirements, the consistency is not an important variable but will be considered a more constant property of the mix.

3. Weight of concrete—While concrete may vary from 80 to 300 lb per cu ft, about 95 percent of the formwork involves ordinary concrete weighing 145 to 150 lb per cu ft. Minor variations in this weight are not significant, and the committee has used the more common value of 150 lb per cu ft in this

report. It should be clearly understood that the recommended pressures should be adjusted to fit the unit weight of the mix in question.

4. Maximum aggregate size—This influences pressure in only a minor way and is subject to too many variations on the same job to warrant inclusion as an important variable.

5. Temperature of concrete mix—This is recognized as an important variable by recent investigators. It affects the setting time of the concrete, and lateral pressures vary inversely with temperature for a given rate of placement. That is, the lower the temperature, the higher the equivalent fluid pressure because the concrete takes longer to set. Available data do not permit any conclusions on lateral pressures for concrete mix temperatures less than 40 F. Most specifications do not permit the temperature of the concrete to go below 50 F when placed in cold weather.*

6. Ambient temperatures—Again temperature is recognized as of major importance. The ambient temperature is often used for a measure of the concrete temperature unless unusual conditions prevail. For example, when cooling water is used to maintain concrete temperature at a minimum, the ambient air temperature cannot be used.

7. Smoothness and permeability of forms—Whereas these were considered important variables by earlier investigators, the use of mechanical vibration in lieu of spading has relegated them to minor importance. In addition, modern form construction and the application of form oil to permit reuse of the forms has further minimized their effect. However, if rough forms are used for narrow (6-in.) walls and slender columns, these variables may become important. The committee recommends the approach given by Schjödt[1] as the most informative.

8. Cross section of forms—See depth of placement, p. 177.

9. Effect of consolidation by vibration—The angle of friction for ordinary concrete lies between 20 and 30 deg and may go up to 35 deg for dry (low slump) concrete. Mechanical vibration can increase this to 50 deg according to L'Hermite. This aids consolidation and should cause a decrease of lateral pressure from the freshly placed concrete. He further states that fresh concrete, which is being vibrated with a 3000-rpm vibrator, acts as a liquid until it is subjected to a pressure of 12 psi (11 ft).

However, tests show that while mechanical vibration consolidates the concrete, this method of vibration results in lateral pressures that are increased 10 to 20 percent over simple spading procedures. Revibration of previous layers should be minimized in normal forms designed according to this recommendation (p. 189) as the resulting increase in lateral pressures from the action of the vibrator in the stiffened concrete can cause failure of standard forms.

10. Placing procedures—Ordinarily, concrete placement consists of 18- to 24-in. layers throughout the entire form consolidated by vibration as noted

*See "Recommended Practice for Winter Concreting (ACI 604-56)," ACI JOURNAL, June 1956, *Proc.* V. 52, pp. 1025-1047.

above. The committee realizes that deeper layers are often permitted or that two of the shallower layers might be made before the vibration is accomplished. Forms designed in accordance with the formula developed by Committee 622 will be adequate for vibration limited to depths not exceeding 4 ft below the top of the concrete surface. Vibration must be used for the purpose of consolidation only, and not for lateral movement of the concrete unless additional bracing or extra strong forms are used. It has been repeatedly found that external vibration, if used indiscriminately, can destroy the strongest form. Some agencies can prohibit its use except when forms are specially designed for such external vibration. Formwork drawings should carry a statement explicitly defining this danger. Careless dumping of fresh concrete adds a temporary, local, impact pressure up to 50 percent, for which the forms need not be designed. The variables noted above all contribute to the lateral pressure, but they may be controlled by simple adherence to good placing practices.

11. Pore water pressure—Some investigators (Schjödt and Terzaghi) state that the pore water pressure is a definite part of the total pressure against the forms. Pore pressure may be computed if the amount of water and volume of voids in the mix are known. Then, the loss through forms is subtracted. Pore water pressure in fresh concrete was measured in a test made by the Norwegian Geotechnical Institute. This is probably the first time such a measurement has been carried out. This variable may well prove to be a better approach to the more accurate resolution of lateral pressures, but the committee is hesitant to recommend it until more basic research has been done.

12. Type of cement—The average portland (Type I) cement has been used by most investigators and will form the basis of the committee's conclusions. Slower setting cements, set retarding admixtures, and sand-rich mixtures result in higher lateral pressures as the concrete remains fluid for a longer time.* This effect can be compensated for by reducing the rate of placement.

13. Depth of placement—Depth of placement under customary methods of placement will be considered in two separate parts: (a) depth of placement in column forms, and (b) depth of placement in wall forms. In this discussion, a column form will be a form with a maximum horizontal dimension of 6 ft or less, and all other forms will be classified as wall forms.

(a) In many types of construction, column sizes are small and concrete is placed throughout the entire height of the form in a short time. The accompanying vibration necessarily extends throughout the full height of the form (which should not be allowed to exceed one story height or 18 ft). If greater heights are placed, an interval of at least 2 hr should elapse after each 18-ft lift prior to continuing the placing. The resulting lateral pressures are much greater than in cases with controlled rates of placement, since high rates of

*For effect of admixtures on concrete properties see "Properties and Uses of Initially Retarded Concrete," by Lewis H. Tuthill and William A. Cordon, ACI JOURNAL, Nov. 1955, *Proc.* V. 52, pp. 273-286.

placement do not permit utilization of the concrete set to reduce pressures. These resulting pressures are essentially fluid and are the maximum possible. Although some authorities recommend a maximum pressure of 2000 psf when concrete is placed (not dumped) and vibrated in columns, the committee feels that with the base of the form held securely in position by wedges and temporary dowels embedded in the floor below, column forms should be designed to resist a maximum pressure of 3000 psf at one-third of the form height above the floor and varying linearly to zero at top and bottom of form. This figure is based on unpublished field tests and observations performed by the U. S. Bureau of Reclamation at Hungry Horse power plant in 1952.

(b) To permit utilization of the concrete set to reduce form pressures, the depth of fresh concrete that can be added in the form at any one time must be limited to avoid the condition of fluid pressure discussed in (a). Therefore, for any form over 6 ft long and for any column form not designed for the pressure given in (a), the depth of placement will be defined as the increment of height that the level of fresh concrete is raised during one pass in the placing operations. This increment of height shall not be greater than the numerical value of the rate of placement or 4 ft, whichever is smaller, to insure that the condition of fluid pressure discussed above does not occur. In this way, the rate of placement, which is recognized by all investigators and contractors as having a direct primary effect on lateral pressures regardless of the total height of the placement, will be considered more important than the depth of placement used in pressure computations for uncontrolled rate of placement discussed in (a). The maximum form pressure for controlled rates of placement shall be assumed to be 2000 psf.

Recommendation

A number of important factors related to form design must be considered in the determination of the variables to be used. To begin with, the determination of the pressures must be straightforward with a minimum of variables and assumptions, so that it can be readily adapted to everyday use and can effect safe, economical formwork. Moreover, it must be borne in mind that the lateral pressure assumptions have an over-all effect on the complete form assembly. This form assembly is subjected to loads and variables beyond the actual pressure of the concrete. Some of these are swelling of form members, uneven tightening of form rod assemblies, construction live loads, and general carelessness in form assembly. All of these tend to decrease rather than increase the strength of the formwork. Moreover, many forms are temporary structures and are usually designed with less safety factor than permanent structures. A marginal form pressure recommendation can, therefore, result in failures of forms at the low safety factors prevailing.

With the above discussion in mind, and after having reviewed all of the variables, Committee 622 found that rate of placement, temperature of con-

crete mix, and the effect of vibration are the most important variables to be used in form designs and can be combined in a basic formula:*

$$p = 150 + \frac{9000R}{T} \text{ (maximum values: 3000 psf for columns and 2000 psf for walls)}$$

where p = lateral pressure in psf, R = rate of placement in ft per hr, and T = temperature of concrete mix in the forms in deg F. The effect of normal vibration has been included in the coefficient preceding R/T. Normal vibration is considered to be mechanical agitation of the freshly placed concrete by internal vibration within about 15 min of time of placement. Further, the formula is based on the following considerations: (a) revibration of previous layers will be minimized, and revibration will be done only when concrete in such layers is in a plastic condition; (b) external vibration of the forms will not be permitted; (c) the unit weight of concrete is assumed to be 150 lb per cu ft and concrete slump is assumed to be 4 in. or less.

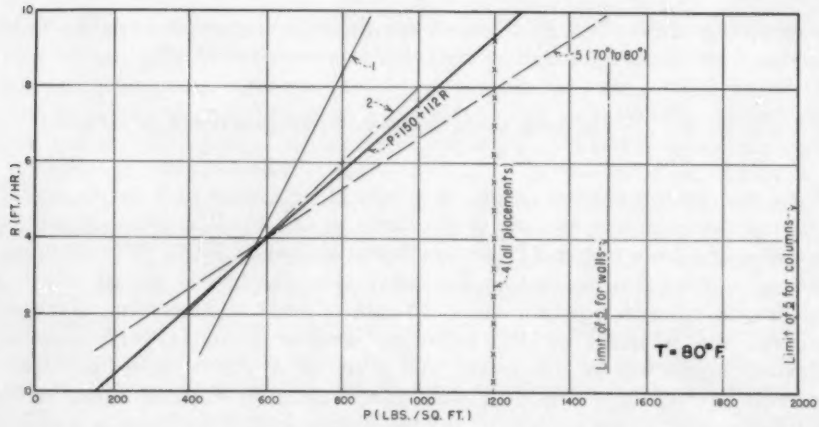
A graphical comparison of the relationship between this proposed formula and the pressure data furnished this committee by most of the major form manufacturers is given in Fig. 1. Separate charts are shown for the main temperature values listed in the manufacturers' data, and the plotted results present an encouraging picture. The proposed formula has the two important practical advantages of being easy to apply and easy to remember.

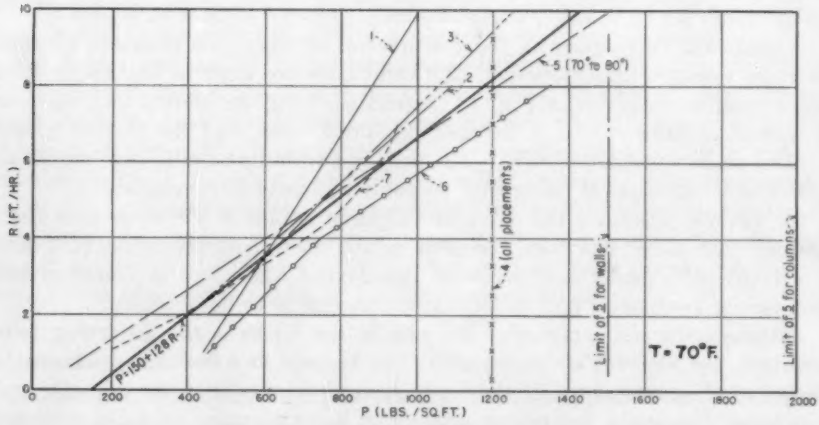
As mentioned above, the effect of vibration is one of the three important factors. Its effect has been included in the coefficient preceding R/T and is a constant. While this may seem paradoxical, the effect of vibration will not vary if consistent, well consolidated concrete is to be obtained.

Although the temperature of the mix in the forms is the governing temperature, the ambient air temperature can be used as a seasonal guide in the design of the formwork. If the form designer knows that the concrete will be placed during the summer months or in a hot climate, he could probably assume a mix temperature of about 70 F and in cold climate, a temperature of perhaps 50 F. In any event, the formwork drawings should show the temperature value and rate of placement used in the design. If the temperature during placement is markedly less than that assumed in the design, such as when the concrete is placed in winter but form design was based upon placement during summer weather, the rate of placement should be adjusted downward as necessary to prevent exceeding the maximum pressure used in the design.

Further discussion of the proposed formula is in order. The curve of pressure due to the concrete alone undoubtedly commences at the origin, goes nearly through the common intersection of the plotted formulas near a rate of placement of 4 ft per hr (the point where most of the test data is available and agreement should be the closest), and then asymptotically approaches a maximum pressure for high rates of placement. Since the pressure value

*See "Conclusions," p. 189, for a modification of the basic formula recommended for rates of placement greater than 7 ft per hr in wall forms.


R (FT./HR.)
P (LBS./SQ. FT.)
P=150+112 R
5 (70° to 80°)
4 (all placements)
Limit of 3 for walls
Limit of 2 for columns
T = 80°F.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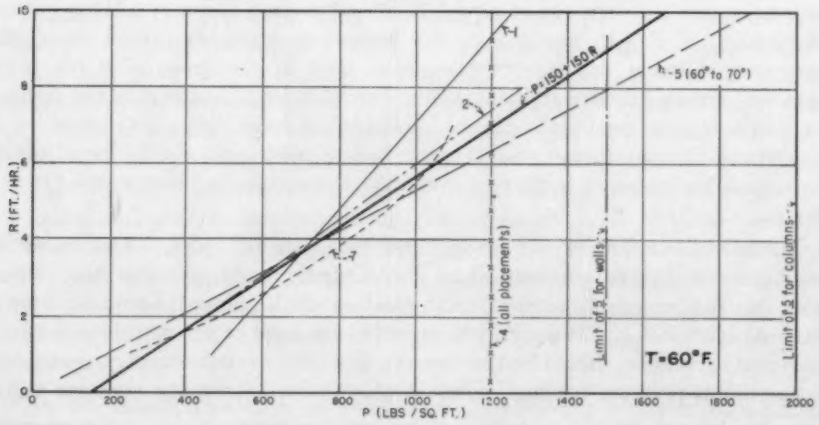
4 (all placements)
Limit of 3 for walls
Limit of 2 for columns
T = 70°F.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4 (all placements)
Limit of 3 for walls
Limit of 2 for columns
T=60°F.

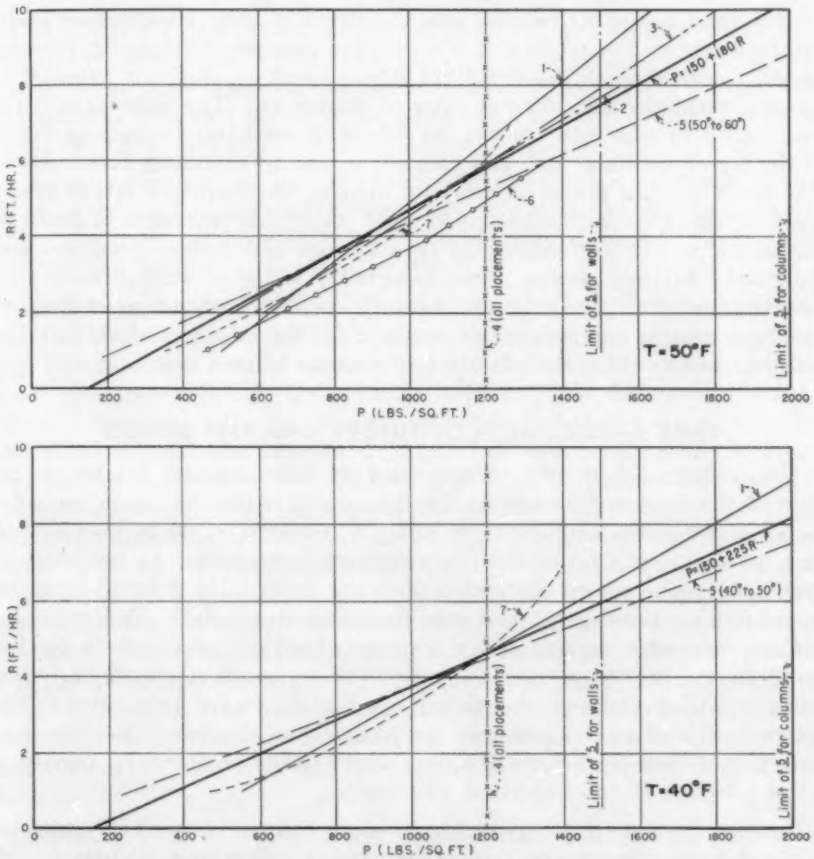


Fig. 1 (left and above)—Comparison of proposed formula for lateral concrete pressures with pressure data from form manufacturers

General equation: $p = 150 + \frac{9000\,R}{T}$

Curves shown on charts have been plotted to include the effect of normal vibration as recommended by the manufacturer:

Curve 1—Williams Form Engineering Corp. (Impact, normal vibration included)

Curve 2—Irvington Form & Tank Corp. (Add 15 percent for vibration)

Curve 3—Symons Clamp and Manufacturing Co. (For reasonable vibration use pressure for rate of placement 1 ft above actual)

Curve 4—Economy Forms Corp. (Steel forms only and design pressure = 1200 psf based on deflection; rate of placement and temperature not mentioned)

Curve 5—Richmond Screw Anchor Co., Inc. (Effect of vibration within 15 min of placement included; based on tests and field observations)

Curve 6—Douglas Fir Plywood Association (Vibration included; based on data by Universal Form Clamp Co., 1946)

Curve 7—The Dayton Sure-Grip and Shore Co. (Add 10 percent for vibration; 30-50 percent for impact; 10 percent for arching when wall is less than 10 in. thick)

used in form design will also include the effects of other construction loads, as previously mentioned, it was felt that the proposed formula for pressure should not go through the origin but should specify a minimum pressure to be used in form design for low rates of placement. The value of 150 psf was chosen to take into account the effects of surcharge loads, revibration at the top of the form, etc., and does not appear to be unduly conservative. The committee has placed an arbitrary limit on the maximum rate of placement for this type of construction of 10 ft per hr (see discussion of depth of placement, p. 177, for pressure to be used with high rate of placement and unlimited vibration) because of the uncertainty in the pressure at the higher rates of placement. The data available indicate that the departure of pressure from the straight line assumption is minor for the range specified, and the practical aspects of the straight line formula were allowed to govern.

PART 2—DISCUSSION OF THEORY AND TEST RESULTS

The bibliography, p. 190, recommended by Subcommittee I after its review of the literature, served as the data in the review of lateral pressure theory and test measurements. In addition, research on the subject carried back to the work of the early investigators (1900-1920). As discussed in Part 1, the problem as it confronted them was appreciably different from the one of today. Essentially, they were pioneering in the field. But more important, they were working with a material whose properties were only generally known, not the concrete of today. Therefore, their measurements of concrete with 8- and 9-in. slumps and hand spading have little value in the determination of lateral pressures for modern-day concrete. Nevertheless, a reading of their work gives a better insight to the problem and permits a better selection of the important variables.

First attempts at plotting the results of all the test data which could be gathered were discouraging. However, it was found that Rodin's paper[5] was an exhaustive study of the same data with which the committee was working and summarizes the results of practically all known investigators prior to 1952. The scatter, resulting from plotting various selected test measurements, is shown in his charts on relation between maximum pressure and rate of placement, head of concrete at maximum pressure, effect of mix proportions, effect of slump of concrete, and effect of temperature of concrete. After reviewing and spot-checking Rodin's paper it was decided that it can well be taken as documentary evidence of the great majority of data and tests made prior to 1952. This is an important step as it presents the varied and scattered data in a convenient working arrangement, and Rodin's efforts should receive proper acknowledgment from the committee. The formulas which he evolved from his study of these data are plotted in Fig. 2.

Since 1952, two other theories have been presented. The first is a series of tests and the subsequent formula developed by the research division of the

Hydro-Electric Power Commission of Ontario.*[17-19] Briefly, they began with the theory developed by Hoffman[4] and evaluated the constants by tests conducted in the field by their research division. These data are also plotted in Fig. 2. The second theory is the subject of a paper by Schjödt[1] in 1955. It is also plotted in Fig. 2.

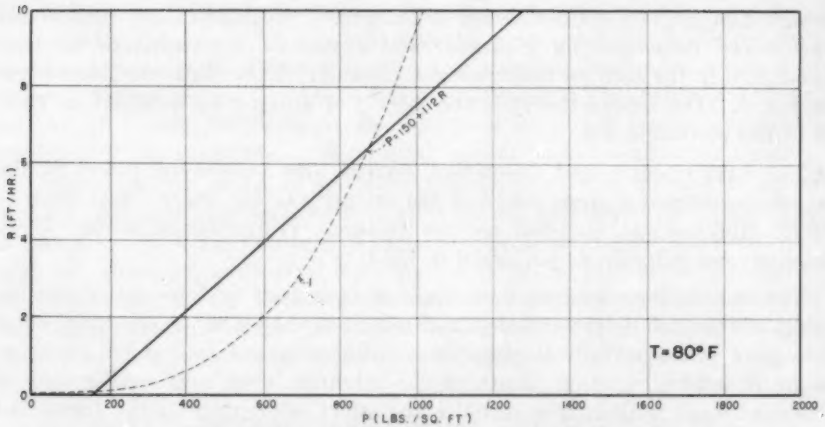
The experimental and theoretical curves now consist of three: Rodin, which is assumed representative of the majority of the known data prior to 1952; Hoffman, as modified by the Ontario Hydro-Electric Power Commission; and Schjödt, as published in 1955.

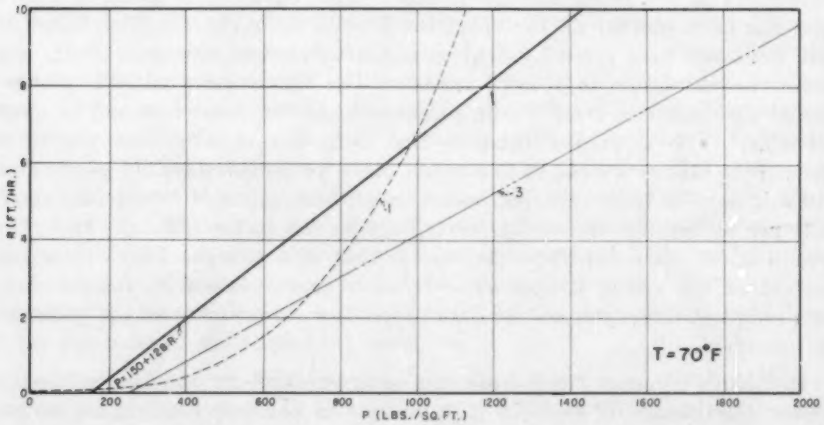
The formula for maximum form pressure as derived by Schjödt includes the effect of the unit weight and depth of mix above the point in question, effective pore pressure, rate of placement, ratio of lateral to vertical pressure, angle of internal friction, depth of the vibrated layer, and setting time of the concrete. Schjödt also includes the effect of friction on the forms but this effect is not being considered here. This equation is given in Fig. 3 and has been plotted on the chart for $T = 70$ F in Fig. 2. The values of the constants were given by Schjödt and are based on a test by Roby (see below for discussion of Roby's values). The temperature of 70 F was selected by deduction from Rodin's discussion of the same test and is questionable. The surprising thing is that Schjödt's equation for maximum pressure is that of a straight line which, when projected, does not go through the origin. The basic equation cannot be used for values of R less than about 1 ft per hr because one of the terms includes the factor $1/R$. If, then, the equation for maximum form pressure is that of a straight line with an intercept on the x axis, it appears to be more practical to solve for the slope and intercept directly from test data rather than to estimate all the variables he considers.

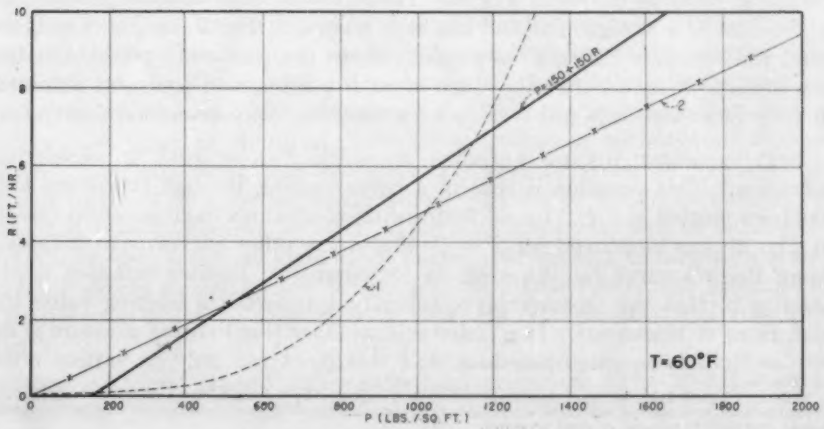
Hoffman's equation, with constants as determined by the Hydro-Electric Power Commission of Ontario (for one rate of placement only), has an exponential form as shown in Fig. 3. The equation of maximum pressure is, again, that of a straight line and has been plotted in Fig. 2; in this case, however, the line goes through the origin. Since the maximum pressure is the one desired, it seems that the slope of such a line could again be obtained directly from test data rather than by evaluating the exponential expression.

Next, consider Rodin's equation in which $P_{max} = 540R^{1/6}$, considering vibration. This equation is that of a curve passing through the origin and has been plotted in Fig. 2 for all temperatures. Rodin's basic equation (given in Fig. 3) was developed for $T = 70$ F and the other curves were obtained using Rodin's curve for the effect of temperature. Rodin's equation is attractive in that the pressure asymptotically approaches a limiting value for high rates of placement. It is debatable whether this limiting pressure is as low as Rodin's equation indicates, and this point can only be settled after

*This work included development of a simple test method suggested by Subcommittee I of ACI Committee 622. To insure that future research results be comparable, the subcommittee will furnish details of the method to organizations interested in research on form pressures.


R (FT./HR.)
P (LBS./SQ. FT.)
P=150+112 R
1
T=80° F


R (FT./HR.)
P (LBS./SQ.FT.)
P=150+128 R
1
3
T=70°F


R (FT./HR.)
P (LBS./SQ.FT.)
P=150+160 R
1
2
T=60°F

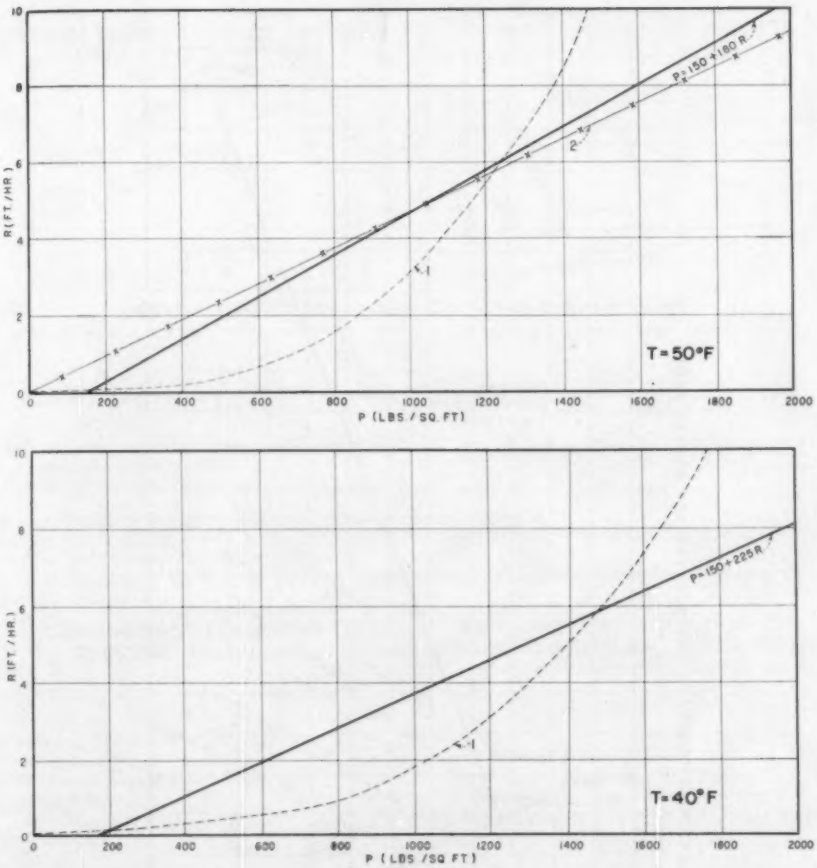


Fig. 2 (left and above)—Comparison of recommended equation for lateral concrete pressures with theoretical and test data

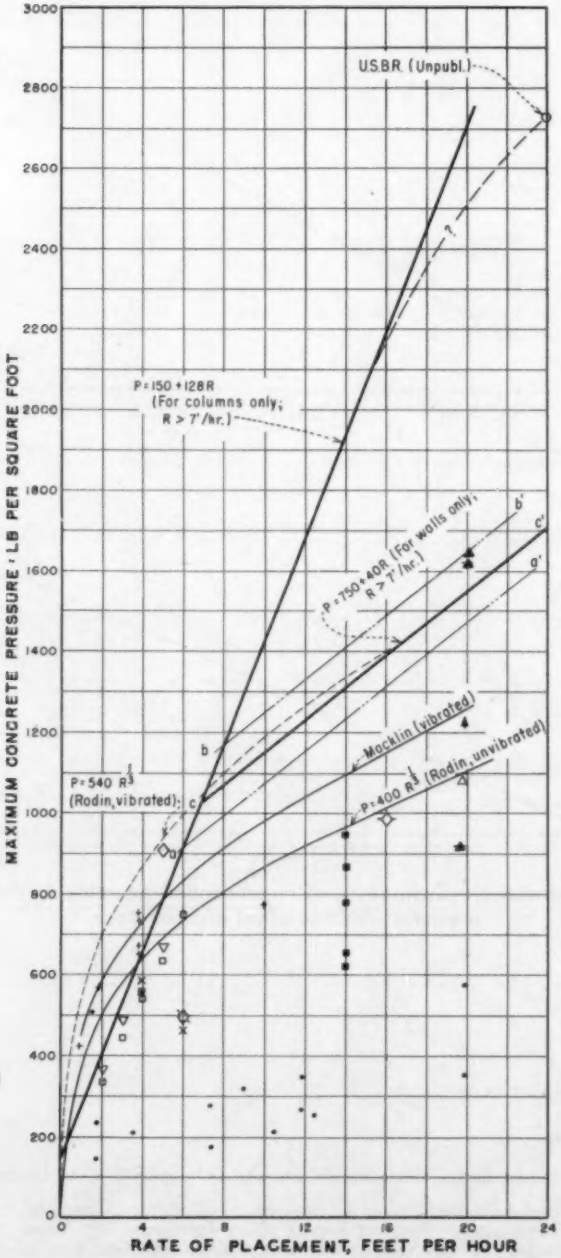
General equation: $p = 150 + \frac{9000\ R}{T}$

Source of data:

Curve 1—Stanley Rodin, "Pressure of Concrete on Formwork" (See Reference 5)

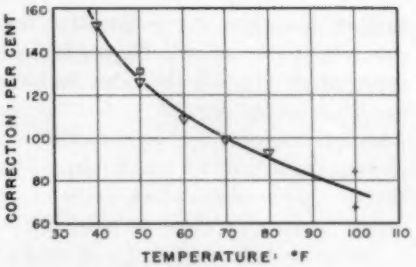
Curve 2—Hydro-Electric Power Commission of Ontario, "Concrete Form Pressure Investigation—Progress Report" (See Reference 18). Based on theory of Rudolf Hoffman with constants determined from field tests where R = 0.88 ft per hr and T = 55 to 60 F

Curve 3—R. Schjödt, "Calculation of Pressure of Concrete on Forms" (See Reference 1)



RELATION BETWEEN MAXIMUM PRESSURE AND RATE OF PLACEMENT (T = 70 F)
(ADAPTED FROM RODIN)

EFFECT OF TEMPERATURE OF CONCRETE ON LATERAL PRESSURE (FROM RODIN)



RECOMMENDED EQUATION OF MAXIMUM PRESSURE (SCHJÖDT)

$$p_{max} = \left[\frac{\gamma\lambda}{2}\left(1+\frac{h_1}{h_e}\right)+\frac{\gamma_o\kappa}{2}\left(1-\lambda-\lambda\frac{h_1}{h_e}\right)\right]\left[h_1+\frac{h_e}{2}\left\{1-\frac{h_1}{h_e}+\frac{\kappa}{\lambda}\left(\frac{\gamma_o}{\gamma+\gamma_o\kappa}\right)\right\}\right]$$

p_{max} = maximum lateral pressure in psf
γ = unit weight of the mix
γ_o = unit weight of water

$\lambda = \tan^2\left(45-\frac{\phi}{2}\right) = \frac{\text{lateral pressure}}{\text{vertical pressure}}$

h_1 = depth to which the effect of vibration reaches down
h_e = (rate of placement) × (setting time of concrete) = height concrete will rise in the form before the bottom part has taken its final set
κ = the ratio of pore water pressure head to height of concrete above the point in question

RECOMMENDED EQUATION OF PRESSURE (HOFFMAN)

$p = \gamma R t\,\lambda_o\, e^{-\alpha t}$ $\left(\text{Note: } p_{max} = \frac{\gamma R \lambda_o}{\alpha e}\right)$

p = lateral pressure in psf
γ = unit weight of the mix
R = rate of placement in ft per hr
t = time in hr

$\lambda_o = \tan^2\left(45-\frac{\phi}{2}\right) = \frac{\text{lateral pressure}}{\text{vertical pressure}}$

α = constant

RECOMMENDED EQUATION OF MAXIMUM PRESSURE (SUBCOMMITTEE I, ACI COMMITTEE 622)

$$p = 150 + \frac{9000R}{T}$$

p = lateral pressure in psf
R = rate of placement in ft per hr
T = temperature of concrete in deg F
Unit weight of concrete = 150 lb per cu ft

LEGEND

Symbol	Investigator	Mix	Slump	Placed by
+	Roby	1:2:3½	+ 7" * 3"	Hand
•	Smith	1:2:4	9½"-11" or sloppy	"
X	Slater and Goldbeck	1:2:1⅓	?	"
△	Teller	1:2:4	△ 7½" ▲ 3½"	"
▽	Maxton	?	?	"
□	Universal Form Clamp Co.	?	?	"
⊙	Bureau of Yards and Docks	7 bags of cement per cubic yard	6½"	Tremie
■	Stanton	?	2"	Vibration
▲	Teller	1:2:4	▲ 1½" ▲ 3½" ▲ 5"	"
◇	Blaw-Knox (Ltr. of 10-30-56 Adjusted to 70°)		◇ 2½" ◇ 5½" ◇ 1⅛"	"

Fig. 3 (left and above)—Comparison of recommended equation for lateral concrete pressure with test data for T = 70 F

further testing. The committee feels that Rodin's equation for maximum pressure is not too reliable for low rates of placement. This is based on the assumption that Rodin was looking for a relatively simple formula that could be easily applied. Also the coefficient of $R^{1/6}$ for nonvibrated concrete was determined from the condition that the curve must pass near a point plotted from tests of maximum pressure with $R = 20$ ft per hr. This equation is an envelope which *includes* all of the plotted points in question and comes reasonably close in most respects. The committee believes this curve is too conservative as it should apply to ultimate strength design of the formwork. For low rates of placement the plotted points are for concrete with slumps of 7 to 9 in. It is doubtful that the maximum pressure for such concrete increases much under the effect of vibration because of the fluid nature of the mix. If such is the case, the equation for pressure with vibration would be tangent to the previous curve for low rates of placement rather than a constant value times the previous curve as assumed by Rodin. This would tend to reduce the pressure at low rates of placement and give a curve which lies closer to the recommended equations.

For higher rates of placement it is also questionable if Rodin has the limiting (i.e., maximum) pressure for the various rates of placement. As mentioned in Part 1, tests performed by the U. S. Bureau of Reclamation gave a pressure of 3000 psf for a rate of placement of about 25 ft per hr at an estimated temperature of 60 F. Using Rodin's curve for the effect of temperature this would mean a pressure of 2730 psf at 70 F, and, for $p = CR^{1/6}$, the value of C would be 935, in lieu of 540.

Fig. 3 contains a reproduction of Rodin's Fig. 12[5] with the chart extended to include the value of 2730 psf mentioned above at the maximum rate of placement of 24 ft per hr. Additional data and the committee's recommended equation of pressure for 70 F ($p = 150 + 128R$) have been plotted on this chart. Since the point labeled "USBR" (unpublished) is approximate only, the exact shape of this curve is not known for the higher rates of placement. In the range of the lower rates of placement, the recommended equation is in good agreement with the plotted data except for Roby's values. Roby's value of 540 psf for a rate of placement of 1 ft per hr appears too high when compared with the data available to this committee.

The equation of maximum pressure recommended by Committee 622 as plotted in Fig. 3 represents the maximum possible pressure for the various rates of placement when vibration is allowed throughout the entire height of the form and is the basis for the recommended pressures in column forms as given in Part 1. However, for wall forms, the maximum depth of vibration has been limited to 4 ft from the top surface of the concrete in the form. It seems reasonable to assume that, for high rates of placement, the maximum pressure would be less than in the case where vibration is permitted through the entire height of plastic concrete. It is felt that the limiting value of maximum pressure under controlled vibration lies somewhere between lines a-a' and b-b' in Fig. 3 for rates of placement greater than 6, or 8, ft per hr.

Line c-c' ($p = 800 + 40R$) agrees fairly well with the upper portion of Rodin's curve, but it departs from the curve of maximum possible pressure at a rate of placement of 7 ft per hr. It is offered for discussion. If future tests prove that the departure from the curve of maximum possible pressure does occur in this general region, the equation of recommended pressure plotted in Fig. 1 and 3 could then be revised.

CONCLUSIONS

The committee feels that the apparent disagreements between the recommended equation of maximum pressure and the formulas discussed above have, in general, been explained. It is felt that this recommended equation, $p = 150 + (9000R/T)$, is sufficiently conservative for all values of R of 10 ft per hr or less and that it may be used for design. Further testing under controlled conditions is needed to more firmly establish the true shape of the equation of maximum pressure for both walls and columns at rates of placement above 6 ft per hr.

In its most general form the proposed formula might be as follows:

$$p = C_1 \frac{(1 + C_2R)}{T}$$

where C_1 is a function of the unit weight of mix and C_2 is some function of the consistency of the concrete. It would appear the p should increase with the increased plasticity of the mix. Assuming that the proposed formula was based on unit weight of mix = 150 lb per cu ft, it would then be as follows:

$$p = 150 \frac{(1 + 60R)}{T}$$

or for columns:

$$p = 150 + \frac{9000R}{T} \text{ [maximum value 3000 psf (see p. 178)]}$$

or for walls and similar work:

(a) with R controlled and less than 7 ft per hr

$$p = 150 + \frac{9000R}{T}$$

(b) with R greater than 7 ft per hr

$$p = 750 + \frac{2800R}{T} \text{ [maximum value 2000 psf (see p. 178)]}$$

For this reason the important data to be collected in future tests in addition to the pressure are the unit weight of mix (lightweight concrete), con-

sistency of mix, rate of placement, pore water pressure, and temperature of mix, all measured under the controlled conditions of mechanical vibration and slump previously discussed in Part 1, p. 178.

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*Committee 622 is presently collecting references on the entire subject of form design for an annotated bibliography.

†"Lateral Earth and Concrete Pressures," by Lazarus White and George Paaswell, *Transactions*, ASCE, 1939, Paper No. 2051, contains a digest of this information.

Discussion of this report should reach ACI headquarters in triplicate by Nov. 1, 1958, for publication in the March 1959 JOURNAL.

Title No. 55-11

From a symposium sponsored by ACI Committee 215

Fatigue of Concrete—A Review of Research*

By GENE M. NORDBY†

SYNOPSIS

Investigations of fatigue of concrete are reviewed starting in 1898 with the work of Considére and De Joly. More than 100 publications on the subject, including those in the foreign literature, were surveyed. The most important investigations are summarized and the salient facts which seem to be emerging from the research are listed. The work has been divided for discussion into six categories: fatigue in compression, fatigue in flexure, fatigue in tension, fatigue of bond, fatigue of reinforced concrete, and fatigue of prestressed concrete.

INTRODUCTION

As the methods of analysis for structures become more exact, the demand for more fundamental information on the behavior of concrete under loads other than static has become more pressing. Highway engineers, of course, have been aware of the need to understand the fatigue of concrete in connection with pavements and reinforced concrete bridges. Structural engineers have become more aware of the need to understand the action of materials under fatigue and impact loadings. There is, however, little data of the necessary precision required for improved procedures, but many investigations which might best be classed as exploratory have been carried out both in this country and abroad.

ACI Committee 215, Fatigue of Concrete, was organized in 1947 to, along with other duties, review available data on the behavior of plain and reinforced concrete under repeated loads, determine the gaps in our knowledge, and encourage research which will provide the information to fill these gaps. The committee has compiled an extensive annotated bibliography which will be published in the near future, and the author utilized it in the preparation of this paper.‡

Interest was shown in the fatigue of metals as early as 1829 when Albert[1] subjected mine-hoist chains to repeated proof loadings. He was followed by a number of investigators associated with railway companies who were motivated by practical problems. Notable among them was Hodgkinson (1849) who was interested in the construction of bridges and Wöhler (1852)

*Presented at the ACI 54th annual convention, Chicago, Ill., Feb. 26, 1958, as a part of the symposium sponsored by ACI Committee 215. Title No. 55-11 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 30, No. 2, Aug. 1958, *Proceedings* V. 55. Separate prints are available at 60 cents each. **Discussion** (copies in **triplicate**) should reach the Institute not later than Nov. 1, 1958. Address P. O. Box 4754, Redford Station, Detroit 19, Michigan.

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‡Other summaries have been published periodically in the past (see References 4-8).

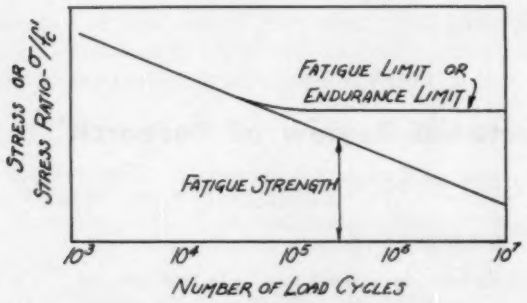

Fig. 1—Typical *S-N* (stress versus log of number of cycles) diagram

who tested axles of railway vehicles. Despite the early interest in metal fatigue, interest in nonmetallics, particularly concrete, lagged for nearly 40 years until Considére[2] and De Joly[3] began tests on mortar specimens. Concrete fatigue investigations, like those on metals, have also been motivated by practical problems, but in studying the fatigue of concrete, engineers have been faced by a dilemma. First, they must solve immediate problems which involve reinforced concrete structural elements. These elements can fail in a number of different ways, i.e., concrete failure in compression zone, diagonal tension failure in the shear zone, fracture of reinforcing steel, and bond failure between steel and concrete. The interrelation between these actions is complex and has defied theoretical explanation even for static loads. Therefore, an even more difficult problem is presented in studying the fatigue of these elements. A second part of the dilemma is the knowledge that the answer to the problem is the study of the fundamental properties of plain concrete in fatigue and the associated fatigue of bond specimens. Yet it is difficult to obtain financial support for such research because of its fundamental character and the length of time between inception of the test and economic returns.

When a material fails under repeated loads, each smaller than a single static load which would cause failure, it is said to have failed in fatigue. Both concrete and steel possess the characteristics of progressive fracture but the fatigue of steel is well-known when compared with concrete and is not a pressing problem except as severe cracks in the concrete affect the fatigue of the steel reinforcement. Consequently, this review will be directed toward understanding what is known about fundamental properties and how these properties bear on the performance of reinforced concrete as indicated by the scattered beam tests.

Fatigue results are usually presented in the form of an *S-N* curve (stress versus log of the number of cycles of load) as illustrated in Fig. 1. If there is a break in the curve and it becomes asymptotic to a line parallel to the horizontal axis, the bounding stress is called an endurance limit or fatigue limit. Most metals have an endurance limit but concrete probably does not. Thus, the curve continues to slope downward as shown. The fatigue strength is the strength for any predetermined number of cycles of load, usually the end point of the curve.

PLAIN CONCRETE

Fatigue in compression

The fatigue of concrete in compression, especially that of the typical compression specimen, is of particular importance to the structural engineer. His long use of compression tests as a quality control as well as a basis of physical properties for theoretical analysis of concrete structures has naturally led to fatigue tests of such specimes and attempts at relating the standard compression strength to the endurance of the material. Unfortunately these tests have been scattered both as to the time and objective of each investigation so that there is difficulty in correlating results. The first investigation on compression was that of Van Ornum[9,10] starting in 1903; similar experiments are still being carried on at Carnegie Institute of Technology, Lehigh University, and Purdue University. During these years the quality of cements obtainable has made possible concrete of much greater strength for a given cement content and thus the relation of older tests to current practice is not clear. The tests were also started before there was widespread acceptance of the standard cylinder so that both cube strengths and cylinder strengths of various sizes are reported, and, as a matter of fact, current use of prism tests and cube tests abroad remain to complicate the issue. The reader should keep these facts in mind when reading this summary of the research.

Van Ornum's tests included compression, beam, and bond fatigue specimens. His first research report included 92 neat portland cement cube compression specimens and eighteen 7 x 7-in. concrete cubes. The cubes were subjected to a repeated load from zero load to an upper limit ranging between 55 and 95 percent of the ultimate cube strength at 4 cycles per min. His conclusion was stated as follows: "Certain observed peculiarities in tests of concrete led to the conclusion that brittle engineering materials (such as stone, brick, mortars, concrete, etc.), of which cement mixtures are a 'fair' type, possess the property of progressive failure or 'gradual fracture' which becomes complete under the repetitions of load well within the ultimate strength of the material." This created wide interest in concrete fatigue, which despite the previous work in fatigue of metals in the 70 years preceding this action had apparently not been contemplated to any extent for nonmetals in this country. He introduced the *S-N* curve previously discussed, which had previously been used in metallurgy research. In Van Ornum's second report[10] he described fatigue tests on 179 prism concrete specimens (5 x 5 x 12 in.). In this case he introduced age as another variable and tested one series after curing for 1 month and another at an age of 1 year. The ultimate strength was 1200 psi and 1580 psi, respectively, which is low when compared with the concretes now in common use. Nevertheless, Van Ornum did discover some interesting features of fatigue action which several other investigators detailed more exactly in the following years. One of the principal features of his research is illustrated in Fig. 2. This illustrates that the stress-strain curve varies with the number of load repetitions. The convex upward curve gradually straightens under repeated load and finally becomes concave upward near failure. The degree of concavity depicts how

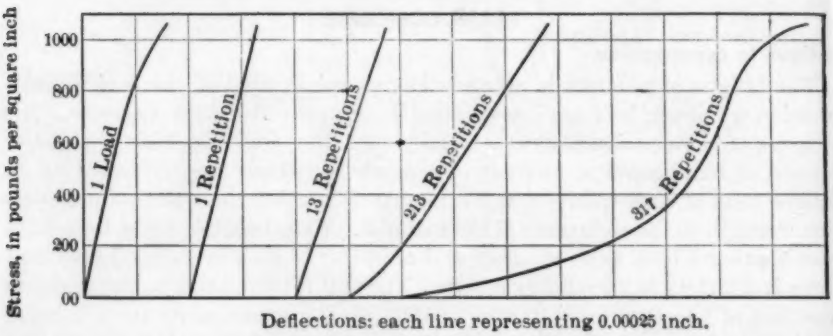

Fig. 2—Stress-strain diagrams of concrete compression specimens at various numbers of cycles of load (Van Ornum[10])

near the test was to approaching failure. An S-shaped curve was actually obtained in some cases. The total effect was noticed only on those specimens stressed above the fatigue limit. A similar reduction in E_c was observed for those specimens stressed below the fatigue limit but in this instance the decrease was stopped after a few repetitions and remained constant as long as the tests continued. This modification of E_c was of particular interest since it affected the modular ratio of the straight-line theory just becoming popular at the time.

Scattered throughout a period of 20 years beginning in 1906 are various studies of the properties of both concrete specimens and reinforced concrete beams under just a few (four or five) repetitions of load to a few hundred. Typical of these are some early tests of Graf[13] and those of Williams.[14] The usual conclusion is that the modulus of elasticity of concrete is increased (or decreased) slightly for repeated loading and the stress-strain diagram becomes a straight line. The results do not represent true fatigue tests and are not further reported in this paper since many times they give an erroneous picture of fatigue results.

About 1910, engineers became more aware of plastic and creep phenomena. Berry,[12] for example, reported the resulting permanent deformations obtained in 8 x 16-in. cylinders. He found that the permanent set increased with repeated load although his tests stopped at 150,000 cycles and were loaded at an intensity of $0.52f_c'$.

One of the most extensive concrete fatigue investigations was carried out by Probst and his associates at Karlsruhe, Germany.[15] Parts of this investigation are summarized in English in two sources[16,17] but important portions are still in German publications and have never been translated. Probst's tests were notable for the precision used in recording the stress-strain diagrams at various stages but the number of specimens was not great.

Mehmel[18,19] was one of the first Probst students to work on the fatigue of compression cylinders. He tested several cylinders with a range of stress from 114 psi to 29.5, 37.5, 47 percent, and others up to and including 70 percent f_c'.

In general, he detailed the action that Van Ornum described in regard to the change in the shape of the stress-strain diagram with number of repetitions. Typical results are indicated in Fig. 3. Some of the work was a continuation of Berry's work on permanent deformations. The Probst group, however, carefully distinguished between permanent deformation and what was called the "remaining" deformation. The remaining deformation was what remained after removal of the load and elastic recovery of the specimen. The remaining deformation was not necessarily permanent since it was discovered that partial recovery would take place during rest periods.

The ratio *d (rem)/d (elast)* [remaining strain/elastic strain] was carefully noted. It was found that elastic strains and remaining strains increase with the number of repetitions as long as a certain critical stress (endurance limit) is not exceeded and that the ratio grows larger with the number of cycles. This critical stress (fatigue limit) was between 47 and 60 percent f_c'. The analogy to strain hardening in metals was also noted since alternating loadings below the critical stress strengthened the concrete. If the upper limit was beyond the critical stress, failure eventually took place by the modification of the stress-strain curve as shown in Fig. 3. The convexity of the curve was a measure of the fatigued state. The same action was noted for higher ranges of stress (1430 to 1856 psi). In every case failure seemed to be similar to static tests. If stressed below the critical, the stress-strain diagram became linear and progressive failure did not take place within the limits of the tests.

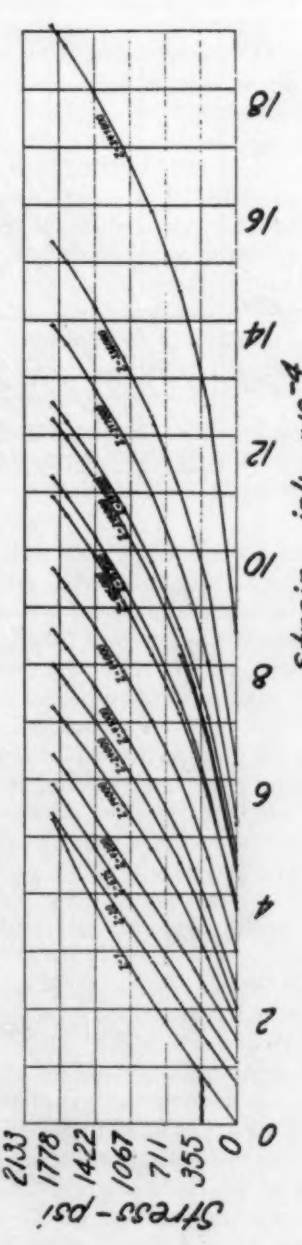

Fig. 3—Stress-strain diagrams of concrete compression specimens at various numbers of cycles of load (Mehmel and Probst[18,19])

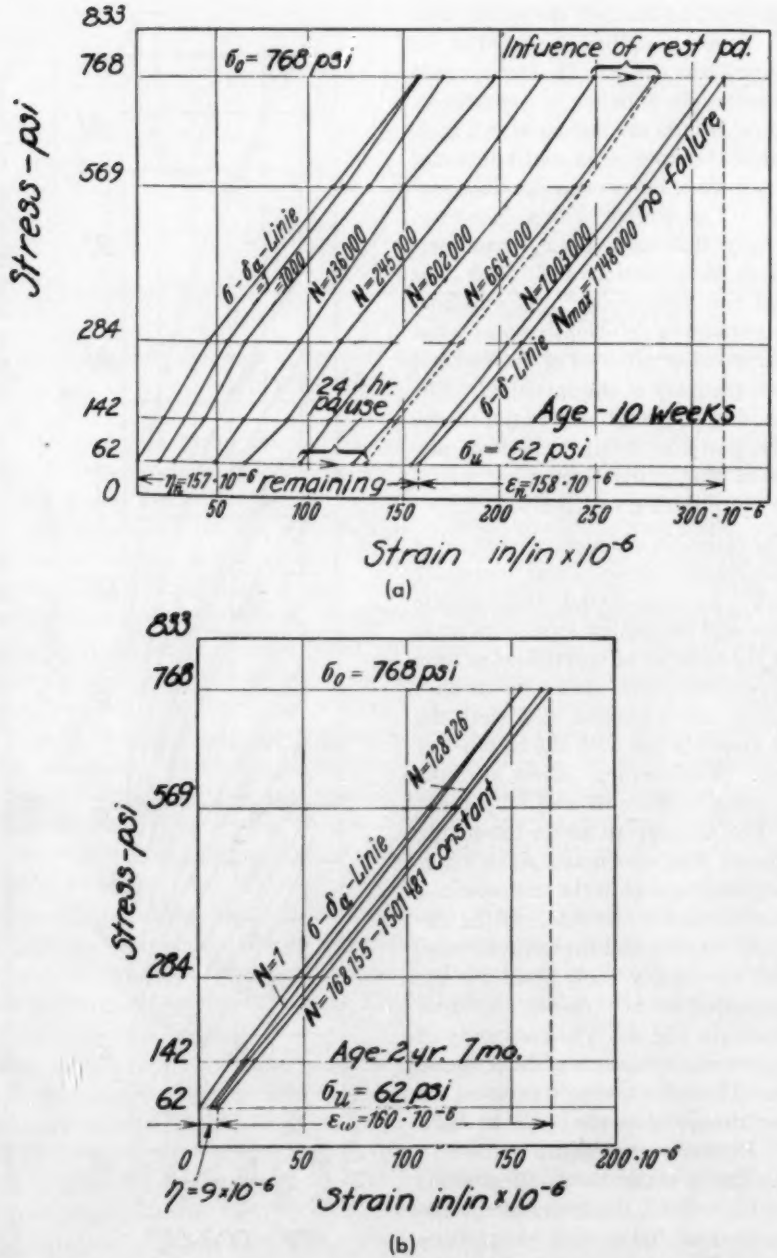

Fig. 4—Effect of age on stress-strain diagrams of concrete compression specimens under repetitive load. (a) Age 10 weeks. (b) Age 2 years 7 months (Trieber[21])

Heim,[20] another Probst student, continued to test compression prisms (6 x 6 x 15.8 in., age: 66-94 days—younger than Mehmels) which indicated greater remaining deformation than elastic deformation after a period of repetitive loading and the remaining deformation did not become constant even after 1,000,000 cycles. Trieber[21] continued this work by reporting the results on cylinders with age of concrete as the variable. One cylinder, for example ($f_c' = 4050$ psi, 10 weeks old), subjected to alternating stresses from 62 to 768 psi had remaining strains equal to the elastic strains after 1,148,000 cycles and stability had not yet been reached. One of the cylinders ($f_c' = 4570$ psi) was 2 years 7 months old when tested and there was no change in the elastic deformation after 128,000 cycles nor in the remaining deformation after 168,000 cycles. This phenomenon is illustrated in the two stress-strain diagrams of Fig. 4a and b. Note the great difference in remaining strains as a result of aging. By comparison the older concrete had clearer elastic properties and the remaining strains were much smaller in the older concrete.

Another investigation at Karlsruhe, reported by Yoshida,[22] examined the change in Poisson's ratio, μ, with repeated load. He found that if $m = 1/\mu$, m was about 7 to 8 after 140,000 cycles of stress from 286 to 1715 psi. This increased to 11 to 12 in the stages close to failure.

In still another investigation under Probst, Ban[23] investigated more precisely the changes in the stress-strain diagram under a repeated load increased by stages. Fig. 5 illustrates a typical result where a cylinder was stressed from 57 to 426 psi for 440,000 cycles, from 57 to 712 psi for 792,000 total cycles, and from 57 to 970 psi to failure at 870,000 cycles. The diagram (Fig. 5) illustrates the hysteresis and change of convex to concave stress-strain diagram. Note that the hysteresis becomes larger in final loadings.

Probably the most penetrating investigation on compression fatigue was carried on at Stuttgart Technical High School by Graf and Brenner.[24,25] Unlike so much fatigue literature, these tests are explicitly described so that they will be of scientific value for years to come. The variables included effect of cross section, speed of testing, mix, curing, range of stress, and age. The tests encompassed about 100 specimens, some of which were tested to 2,000,000 cycles of load without failure. The concrete strength ranged from 2900 to 4600 psi.

One of the unique features of this report was the introduction of the "modified Goodman diagram" to concrete fatigue (Fig. 6). The diagram had been previously used extensively for fatigue of metals. The diagram illustrates the effect of range of stress on concrete fatigue by relating the fatigue strength to the maximum and minimum stress to which a specimen is subjected. One enters the diagram from the left at the minimum stress to an intersection with the 45-deg line, thence vertically to an intersection with the curve and back to the ordinate scale which indicates the maximum stress which can be endured. Thus, the vertical intercept between the 45-deg line and the upper curve specifies the range of stress which can be taken repetitively for a specified time. This time is usually taken as infinite for metals but Graf and Brenner

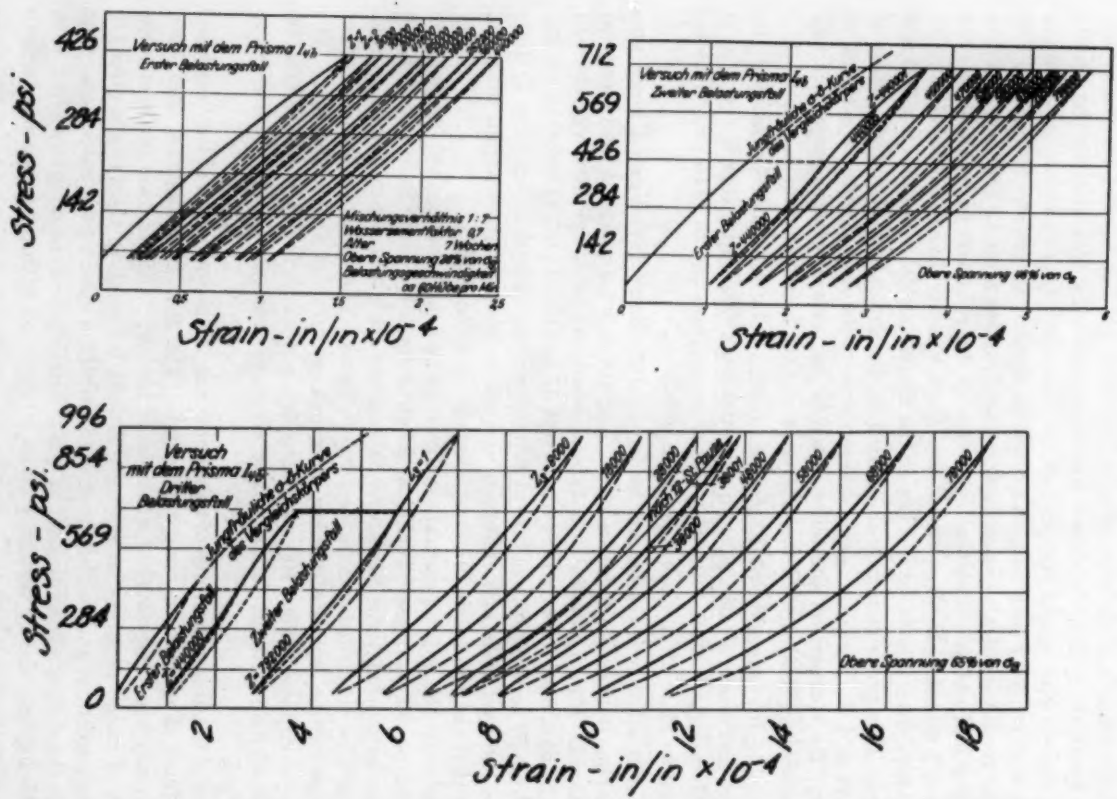

Fig. 5—Effect of three stage increase of repetitive load on stress-strain diagrams of concrete compression specimens (Ban[23])

constructed theirs on the basis of 2,000,000 cycles of load. This technique yields valuable design information and more of these curves should be established on 10,000,000 cycles of load repetition and a greater number of specimens.

Another interesting feature of the tests was the recording of cracks on the surface of the specimens during the tests. It was noted that cracks formed on the surface after a few hundred repetitions. However, the crack did not always result in rapid failure. In one case, for example, a crack formed at 900 cycles but the prism had not failed at 2,245,000 cycles.

As to speed of testing, there did not seem to be any significant difference between results obtained at 260 and at 450 cycles per min. However, there was ordinarily a slight decrease in the endurance for rates as slow as 10 cycles per min. This would be in accord with the observations of Probst that the remaining stresses were larger for slow cycling.

A slight decrease in the fatigue strength expressed as a percent of the ultimate strength was noted for both increase in the water-cement ratio and cement content. A percentage-wise decrease was also noted for an increase in ultimate strength but the absolute magnitude of the fatigue limit increased.

Fatigue in flexure

It seems to be the consensus of highway engineers that the failure of concrete pavements by cracking is due to the repeated applications of stress. This has been the principal motivation for carrying out fatigue tests on both plain concrete modulus of rupture specimens and similar specimens with light reinforcement similar to that in highway pavements. The results of such tests are vitally needed for the optimum design of airport runways and taxiways besides road pavements, as well as understanding the formation of tension cracks in reinforced concrete beams.

The first tests for flexure specimens were reported by Féret[11] in conjunction with a broader investigation encompassing plain and reinforced mortar beams,

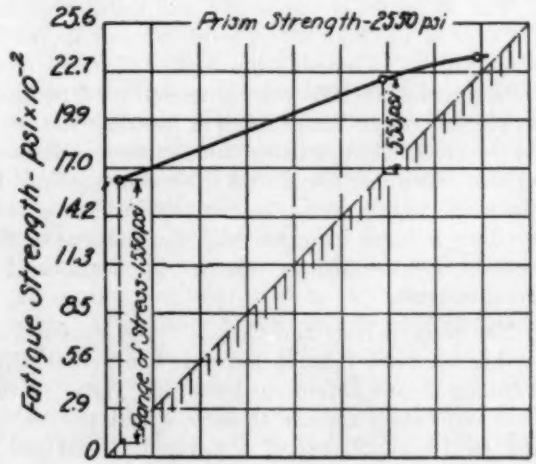

Fig. 6—Modified Goodman diagram for concrete compression specimens (Graf and Brenner[24])

Fig. 5—Effect of three stage increase of repetitive load on stress-strain diagrams of concrete compression specimens (Sah

cubes, and cylinders in 1906. Although these experiments contained some specimens similar to later investigations, they are only of historic interest.

Almost simultaneously, two extensive investigations on flexural fatigue were carried out by Purdue University (1922-24) and the Illinois Department of Highways (1921-23). It had been noted in Illinois highways that failures occurred in many cases only after many years service and that the wheel loads were less in magnitude than the static ultimate loads of the slab. Thus, the Illinois tests reported by Clemmer[27,28] were aimed at duplication of the actual conditions occurring at the vulnerable corner of the pavement slab. Clemmer built an ingenious machine which held seven modulus of rupture specimens (6 x 6 x 36 in.) cantilevered from a central hub. In between the specimens a fill of blocks completed the circular test table. In an effort to duplicate the actual conditions exactly, the load was applied through a pair of rubber-tired wheels which revolved about the central hub. The axle was revolved at 20 rpm to give 40 load applications per min. The failure of the fatigued beams was compared with static ultimate stresses of companion specimens when computing the fatigue effect.

In the first tests 15 beams (1:2:3.5 concrete) were loaded so that the stress was 50 percent of the modulus of rupture (f_t). This load was endured for 1,130,876 cycles without failure. The load was then increased to 61 percent f_t and seven specimens failed at from 17,000 to 200,000 cycles; the majority at under 50,000 cycles. A number of specimens remained intact after a total of 1,540,000 cycles and the load was increased to 70 percent f_t. The remainder failed at 1100 to 91,000 additional cycles of load. As beams failed, additional specimens were installed in the machine; thus, it was possible to compare beams with various stress histories. By this process, Clemmer noted that beams which had been subjected to previous stress histories could resist a greater number of applications of load at an increased stress intensity as long as the first stages of loading were below some critical value (fatigue limit).

The 30 beams of the second and third series failed at less than 5300 cycles because of the high stress level (62 and 70 percent of the ultimate flexural strength). The fourth series is of particular interest. In this case, the beams all endured 2,000,000 cycles at 48 percent f_s without failure; this was increased in stages until all beams failed at less than 700,000 additional cycles at a stress 54 percent f_t. The maximum cumulative applications were greater than 3,000,000 cycles. Clemmer found that the endurance limit is between 51-54 percent of the static failure stress for concrete in flexure, that the number of cycles to produce failures increases with the richness of the mix, and that fatigue resistance is increased by repeated applications at a stress level below the endurance level.

The tests of Hatt and Crepps[29-33] at Purdue University were also inspired by highway slab failures and were concurrent with those of Clemmer. There were significant differences however. Hatt felt that the 40 cycles per min of load application and the absence of rest periods of the Illinois tests violated the service conditions of a pavement, and felt that it was impossible to

duplicate the exact road conditions in the laboratory. Thus, he selected a simple flexure test for his experiments. He also selected a speed of 10 cycles per min for his tests and halted his tests overnight and on weekends to introduce rest periods. The specimens were similar (4 x 4 x 30 in.) to those of the Illinois tests and were tested as cantilevers in a special machine which loaded the specimens by weights alternately lowered on the opposite ends of a cross member fastened to the end of the specimen. This produced reversal of stress. Several mixes were used: 1:2 mortar; 1:1½:3, 1:2:3, and 1:2:4 concretes. The fatigue strength of the test members was compared with that of an unfatigued companion specimen tested to static failure. About half the specimens contained 0.33 percent mesh reinforcing placed ⅜ in. from the tension face to test the effectiveness of small percentages of steel reinforcement in preventing cracking.

Strain measurements were taken at the surface of the beams in most cases. A typical fatigue specimen showed a progressive deformation in the extreme fibers in a fairly uniform manner until about 100,000 cycles of load when several visible cracks appeared, after which the test continued for 35,000 cycles before failure. Dry mortar beams tested in fatigue at 6 months yielded values of unit extension of 0.00015 to 0.00016 while similar concrete beams tested under progressive loading showed up to 0.00017 unit deformation at a stress intensity of 54 to 55 percent f_t. The Purdue tests tended to corroborate the Illinois tests; the fatigue strength was nearly the same. Age of specimen was an added variable. The tests indicated that an endurance limit of 54 to 55 percent f_t could be expected for mortar specimens over 6 months old, and 50 to 55 f_t for specimens 4 months old; but an endurance limit could not be established between 40 and 60 percent f_t for 28-day specimens. Rest periods were found to have only a temporary effect; however, an accidental shutdown gave a rest of 5 weeks to one specimen which permitted almost complete recovery of a specimen on the verge of failure.

The raising of the endurance limit by conditioning of the specimen by stressing below the fatigue limit was also verified. Such conditioning would raise the endurance limit by 5 percent f_t. Tests on a special series of saturated mortar specimens indicated a fatigue limit of 33 percent f_t could be expected.

For the reinforced beams the investigators concluded that small amounts of reinforcement will increase the extensibility of concrete and that failure will not take place when the limit of extensibility is reached as determined by the plain concrete beams. Reinforced beams (0.33 percent steel) had 36 percent greater extensibility, 30 percent greater static strength, and could resist three times the number of load applications.

About 1943, Williams[34] of Stanford performed one of the few sets of tests on lightweight aggregates. The tests were motivated by the use of two aggregates—Haydite and Gravelite—in concrete bridge decks. The specimens were modulus of rupture specimens similar to those used in the Purdue tests although the rate of loading was somewhat faster, 15 cycles per min. Complete reversal of loading was used. The fatigue limit for the Haydite after 1,000,000 cycles of load (30 to 84 days old, $f_c' = 4030$ psi, $f_t = 440$) was approx-

imately 50 percent of the ultimate static strength f_t. For another mix (28 to 83 days old, $f_c' = 3020$ psi, $f_t = 390$) the fatigue limit of 40 percent f_t was established. For the other aggregate, Gravelite ($f_c' = 3100$, $f_t = 556$), a fatigue limit of 40 percent at 100,000 cycles was found. It was apparent from the test that failure often occurred at a point where a weakened piece of aggregate was at the tension surface, which gave the starting point for a crack. Other times cracks developed at many irregularities like holes and air pockets which may be characteristic of harsh mixes.

Recently a more comprehensive investigation has been started at the University of Illinois under Kesler.[35] The first phase of the research was directed toward finding how speed of testing affected the flexural strength of the concrete (1:2.84:4.14, $f_c' = 3600$ psi, or 1:1.84:3.25, $f_c' = 4600$ psi). One hundred modulus of rupture specimens (6 x 6 x 64 in.) were tested in flexure with third-point loading at 70, 230, and 440 cycles per min. The resulting fatigue strengths were compared with the average static strengths determined from flexure tests made on the broken parts of the beam after fatigue failure, thus eliminating variances from specimen to specimen in this comparison. The average fatigue strength at 10,000,000 cycles for all beams tested was approximately 64 percent of the static ultimate strength. There was, however, no evidence that an endurance limit even existed for concrete, as shown in Fig. 7 representing data from a typical series of tests. The line continues to

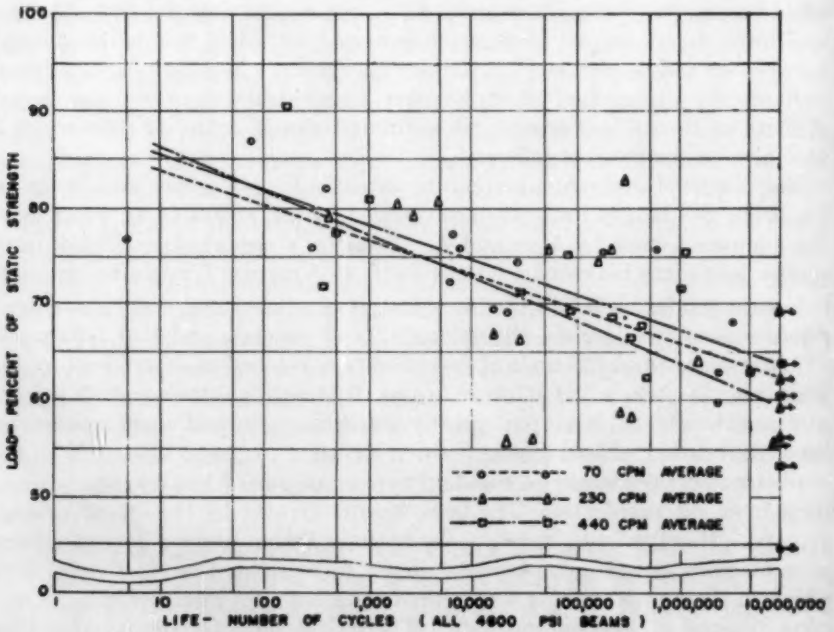

Fig. 7—Effect of speed of loading on fatigue strength of flexural specimens (Kesler[35])

slope downward even at the 10,000,000 cycles limit. Kesler also concluded that speed of testing between 70 and 440 cycles per min had negligible effect on the fatigue strength of plain concrete with sound aggregate. The tests are continuing and other variables will be investigated, some of which are reported in this issue of the JOURNAL.

The results of Kesler are supplemented by Le Camus[26] who proposed a fatigue limit of 68 percent for 1,000,000 cycles of load. The tests were on six inverted T-sections with reinforcement in the compression stem.

Fatigue in tension

Only the early experiments of De Joly[3] reported in 1898 give any information on pure tension fatigue. He reported tests on tension briquets at ages of 2 to 20 days at frequencies of 26 to 92 cycles per min, and determined a fatigue limit of approximately 50 percent of the static ultimate strength. He also found increase in frequency of application reduced the repetitions for failure and that rest periods appear to permit recovery from fatigue effects. Again these tests are of only historical interest. Since tension failure is closely related to the fatigue of plain concrete flexure specimens it may be assumed however that the conclusions of the preceding section apply equally well here.

Fatigue of bond

The mechanism of bond failure has defied analysis since the early stages of development of reinforced concrete. Now prestressed concrete has added further complications to the problem. Various types of pull-out specimens utilizing concretes of various types and both smooth and deformed bars with a multitude of patterns of corrugations have been tested without obtaining a real understanding of bond. As a result, fatigue investigations on this phase of concrete properties cannot progress beyond the crude testing state. Nevertheless, several of the early investigators including Van Ornum,[10] Kirsch,[36] Withey,[37] and Abrams[38] accepted the challenge. Their tests are of little value today but provided the engineers in 1910-20 with test results which gave them faith in their materials. The tests ranged from Van Ornum's impact tests on bars embedded in prisms to Withey's repetitive loading of beams cut away to completely expose the bars except for portions at the ends. The tests were repeated for less than 10 cycles to a few thousand repetitions and resulted in allowable stress limitations. Withey, for example, concluded that bond stresses in excess of 50 psi for round rods should not be allowed and that rust substantially aided bond (83 percent increase over clean bars). A corrugated bar was loaded over 100,000 times without failure with a maximum bond stress of 76 psi. Kirsch, on the other hand, concluded that rust hindered bond. His tests were limited to 50 cycles after which pull-out tests were made.

More recent tests by Lea,[39] Muhlenbruch,[40,41] and Le Camus[26] are of more value. Lea established a fatigue limit for ½-in. round bars in pull-out tests using small prisms so arranged to have the concrete in tension. The bond fatigue limit was about 300 psi regardless of whether the bar was subjected to

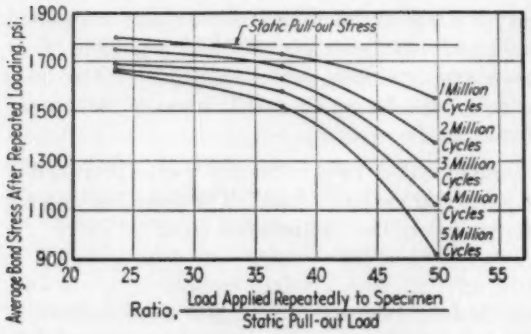

Fig. 8—Effect of repetitive load on ultimate bond stress as determined from pull-out specimens (Muhlenbruch[40])

stresses from zero to a maximum tension or completely alternating stresses. The static pull-out strength in each case was 610 and 685 psi, respectively.

Muhlenbruch completed the most extensive investigation to date. He used pull-out specimens 5 x 5 in. to 10 in. long (concrete, 1:1.5:3.0 by volume, 7.5 gal per sack) with a $\frac{5}{8}$-in. deformed bar embedded 3 to 10 in. They were tested at various intensities up to 5,000,000 cycles of load and the rate of loading was either 26 or 73 cycles per min. The results are summarized in Fig. 8; the average bond stress developed was substantially decreased due to the repeated loads. The reduction was as great as 50 percent when repeated loads of a magnitude 50 percent of the static pull-out strength were applied for several million repetitions. The performance of deformed bar was superior to that of the hot-rolled undeformed bar. For example, the static pull-out strength of a hot-rolled bar was reduced 44 percent as compared to 31 percent for a deformed bar when both were subject to 3,000,000 repetitions of load in one case.

The tests of Le Camus[26] gave higher values. Based on five push-out tests on 30-mm diameter round bars he concluded that the fatigue strength for 1,000,000 cycles was 69 percent of the ultimate push-out strength.

It seems that results on bond are divergent among the various investigators.

Summary of results for plain concrete

The following summary should not be considered as final conclusions since the data upon which they are based are scant and many of the early results are questionable because of the cement improvement of recent years. They do, however, represent the salient facts which seem to be emerging as a result of the exploratory investigations to date.

1. The fatigue limit of plain concrete, subjected to repeated compressive loads for a range of stress from zero to a maximum compression, is 50-55 percent of the ultimate crushing strength. All investigators are in substantial agreement on this.
2. The fatigue limit of plain concrete subjected to repeated flexural loads is about 55 percent of the static ultimate flexural stress although there was a variance from 33 to 64 percent, depending on other variables, such as age, moisture content, curing, and aggregate.
3. The fatigue limit for concrete in tension is about 55 percent of the modulus of rupture.
4. It is doubtful that concrete possesses an endurance limit at least within 10,000,000 repetitions of load.

5. Under repetitive load the modulus of elasticity changes in various ways depending on the intensity of load. The secant modulus decreases with repeated load; the slope of the stress-strain curve may decrease in the lower part of the curve and increase slightly in the upper portion to become concave upward (as indicated in the illustrations).

6. Concrete possesses a property similar to strain hardening in metals. Loading repetitively at less than the fatigue strength resulted in raising the fatigue strength and/or stiffening of the specimen.

7. Age and curing have a decisive effect on the fatigue strength. Inadequately aged and cured concrete is less resistant to fatigue than well-aged and cured concrete.

8. The effect of saturation is unknown; however, one investigator reports a fatigue limit as low as 33 percent of the ultimate static strength.

9. Rate of testing between 70 and 440 cycles per min has little effect on fatigue strength. Slow rates (about 10 cycles per min) seem to decrease the fatigue strength. The creep phenomenon is so interrelated with long-time fatigue loading that it is difficult to separate the effects of the two.

10. Rest periods seem to increase the endurance of concrete although tests results are scant.

11. Most of the permanent (remaining) deformation takes place in the early stages of the test, usually the first few thousand cycles. The "rate of permanent strain increase" decreases with the number of cycles of load. The total strain does, however, continue to increase slightly until failure for stresses above the fatigue strength but reaches a maximum for stresses below the fatigue strength. For properly cured and aged concrete the strain stabilizes at fewer load cycles than for young concrete.

12. Fatigue strength decreases slightly with leaner mixes and higher water cement ratios (data not extensive).

13. As the range of stress is decreased the upper limit of the stress (fatigue strength) is increased substantially. This phenomenon can be represented by the modified Goodman diagram.

14. As regards bond, little can be said except fatigue failures are possible at loads less than 55 percent of the ultimate static pull-out strengths. Results have been erratic.

FATIGUE OF STRUCTURAL COMPONENTS

Reinforced concrete

The fatigue investigation of reinforced concrete commenced concurrently with the research on properties of plain concrete in fatigue. (A selection of cross sections of beams tested by the various investigators is shown in Fig. 9.) Much of this early work was directed toward verifying the modular ratio and the straight line theory for a few dozen cycles of load. Almost all of the fatigue tests have been directed toward beams and research on fatigue loading of columns has been limited to a few dozen repetitions of load (Withey,[43] for example).

Feret[5] was probably the first to examine reinforced concrete structures. He included short columns with spirals only, as well as some small mortar beams. The loads ranged from 50 to a maximum of about 5000 cycles. Other tests of Jarvis,[44] Talbot,[45,47,48] and Hatt[46] involved beams loaded less than 100 times and are disregarded in this review.

Since fatigue tests take so much time, often the early reports were limited to the performance of two or three beams. Berry,[49,50] for example, tested three

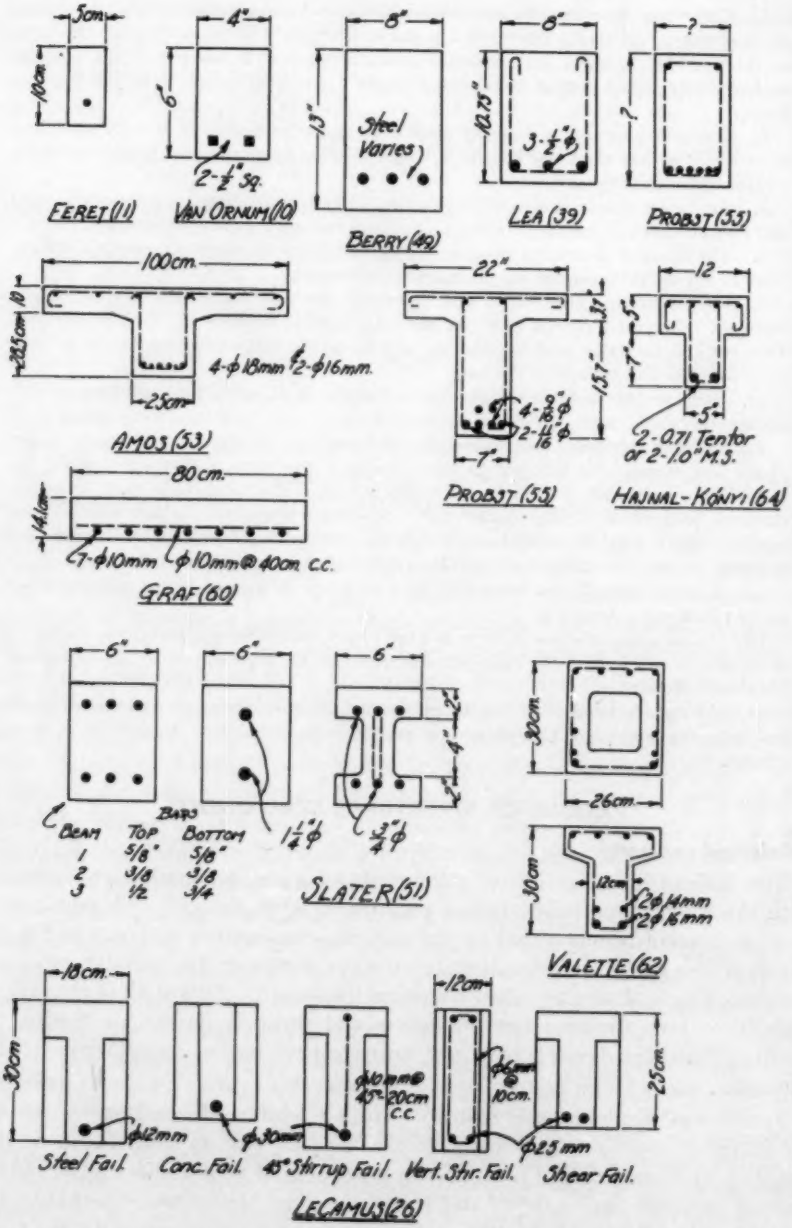

Fig. 9—Cross sections of some of the reinforced concrete beams tested under repetitive loading

beams, one to a maximum of 1,140,000 cycles (13 cycles per min, cube strength 2300-2700 psi, corrugated and diamond bars). He paid particular attention to the permanent set which accumulated under repeated load. However, he found no significant effect on ultimate strengths and deflections at failure although progressive cracking occurred up to 1,000,000 cycles of load. Most of the permanent set occurred in the first few thousand repetitions and increased with the intensity of load and number of cycles. No fatigue failures occurred and beams were tested to failure after the fatigue tests.

Van Ornum[10] was the first to extensively examine reinforced concrete beams. He tested 59 beams at an age of 1 month and 23 at an age of 1 year. These were compared with a number of unfatigued specimens. Van Ornum noted that the beams failed through the development of a tension crack and sometimes diagonal tension crack which seemed to appear gradually and increase in size during the fatigue test. When penetration of the crack progressed far enough the beam failed by steel fatigue. In those beams which failed through formation of a diagonal tension crack, it appeared that gradual progressive destruction of the bond originating near the center of the beam and moving toward the ends caused failure.

During World War I, the government was engaged in building concrete ships. An extensive investigation was carried out at that time on typical structural components by the National Bureau of Standards.[50,51] Five beams of various cross sections were tested aimed at investigating deterioration by grinding and abrasion at cracks. Four of the beams failed by fatigue of the reinforcing steel between 59,000 and 710,000 cycles. The range of stress in the reinforcement was from zero to a maximum of from 17,400 psi to 22,800 psi in these fatigue failures. The fifth beam resisted a maximum stress of 11,000 psi and did not fail at 2,000,000 cycles and the test was discontinued. The T-beam failed first in the stirrups and finally failed in tension of the longitudinal reinforcement.

The effect of corrosive atmospheres on the reinforcement in concrete beams has been of concern from the early days since the cracks allow corrosive agents to penetrate into the beam. To investigate this phenomenon, Amos[52] tested a total of nine beams in three different environments: (1) air, (2) air and water vapor, and (3) air, water vapor, and flue gas. The beams were loaded with two-point loading so that a range of stress from zero to 1000 kg per sq cm (14,230 psi) was imposed on the steel. The loading was at 24 cycles per hr or 20 cycles per min. One of the tests took 3¾ years to complete and 7,500,000 cycles were applied. The number of cracks in a beam increased from approximately 10 to 30 in the first year and virtually no change took place in the final years. None of the beams was adversely affected by the corrosive atmospheres or the load cycling.

Probst[20,55,57] and his associates (Heim and Treiber) have contributed considerably to the knowledge of reinforced concrete beam fatigue in addition to their work on plain concrete. Probst's interest dates from 1907 when his initial report[56] appeared. The literature is somewhat confusing, however,

since his work has been summarized several times in texts[6,17,55] and appears in both German and English[16] journals, and it is not clear how much new work is presented each time. Both T-beams and rectangular beams were tested. The reinforcement included a variety of different bent bars and/or stirrups. As in his plain concrete experiments he paid particular attention to the remaining strains and deflection. He noted that beam cracking was not different from that obtained in a static load. Due to crack penetration and plastic deformation in the compression zone the steel tension increased under the same repeated load, for example, from 16,360 to 20,770 psi in one beam. The width of cracks was less than 0.031 in. after several million cycles of load—0.023 in. elastic and 0.0078 in. remaining deformation. It was noted that aged beams did not permanently deform as rapidly as young beams did; the action was similar to that of plain concrete. He concluded that repeated loads below some critical value (fatigue strength) do not affect the ultimate carrying capacity of the beam and that cracks breathe (open and close) as long as the elastic limit of the steel is not exceeded.

Graf[60] has also recorded remaining and elastic deflections for three large slabs which confirm Probst's observations. In general, they point out the importance of creep and shrinkage, and that they are involved inseparably with the fatigue phenomenon.

One of the most extensive tests on reinforced concrete beams was reported by Saliger[58,59] in Austria. Thirty-two T-beams, 2.7 m long with 0.56 and 1.4 percent reinforcement, and using four types of steel (ST-37, ST-55, ST-80, and Isteg) were tested. The bar sizes ranged from ½ to 1 in. diameter and the cube strength of the concrete was 4700 psi average at an age of 400 to 500 days. The beams were subjected to 1,000,000 to 3,000,000 cycles of load such that the steel stresses were 10 to 30 percent higher than ordinarily allowed in design. The reports available are only summaries, however, and do not give the full details of the tests. Several observed features are worth noting: (1) cracks opened and closed during fatigue loading; (2) the high strength bars performed better for the same load; (3) rest periods meant recovery of the remaining deflections; (4) beams became stiffer as a result of repetitive load and there was elastic recovery at all times; (5) the carrying capacity was not impaired; and (6) Isteg steel may allow greater design stresses for the concrete because it did not elongate and allow crack penetration as rapidly as other steels.

Lea[61] in England has reported a series of eight beams reinforced with three ½-in. diameter smooth bars. These tests are of interest because enough duplicate specimens were tested at various loads to establish a fatigue limit for the particular beam. The tests were carried out to a limit of 11,000,000 cycles for which the fatigue strength was 70 percent of the ultimate load. The method of failure for each beam was typical—a crack progressed toward the top of the beam resulting in excessive elongation of the steel and crushing of the concrete.

Le Camus' work was unique in that his beams were designed in such a way as to make them critical in the compression zone, in the longitudinal reinforce-

ment, in 45-deg stirrups, vertical stirrups, or concrete shear. To do this the beams were of the mushroom type (champignon design) so that the beam is reduced in width to provide a critical section for each action. A variety of flared beams in both rectangular and T-sections were used as indicated in Fig. 9. A total of 25 beams were tested in fatigue in addition to static specimens. The following fatigue strengths based on 1,000,000 cycles were determined (expressed as a percent of ultimate static load in each case): longitudinal reinforcement, 54 percent; concrete compression, 60 percent; 45-deg stirrups, 41 percent; vertical stirrups, 35 percent; and concrete shear, 42 percent. Probably the most important feature of the tests was the low fatigue limits obtained for the concrete shear and stirrup reinforcement cases.

The use of reinforced concrete for railroad structures has always been suspect. The reason is probably that engineers feel railroad bridges ordinarily receive more vibrations and repetition of loads than other types. Valette[62] investigating bridge components for French railways tested three typical members under two-point loading at 500 cycles per min. Two beams failed by fatigue of steel. No wear of concrete nor loosening of the concrete around the cracks was evident during the tests. He concluded that mild steel reinforcement possesses the safety prescribed for all kinds of rolling or stationary loads.

Recent work has been done by Orbom[63] and Hajnal-Kónyi[64] on high grade steel reinforced beams. It appears that such steel may have a decided beneficial effect because of its higher endurance limit; however the latter tests are still in progress.

In summary it may be said that there is far from a complete picture for fatigue of reinforced concrete structures. Reinforced concrete beams may fail either in steel or concrete and the concrete failure in turn may be in bond, diagonal tension, or flexure. The fundamental actions are not understood as yet themselves; however, certain features reappear in many of the investigations reported.

1. Most failures of reinforced beams were due to failure of the reinforcing steel. The failures seemed to be connected with severe cracking and the possible stress concentration and/or abrasion connected with these cracks. Beams critical in longitudinal reinforcement seemed to have an endurance limit of 60-70 percent of static ultimate strengths for 1,000,000 cycles.

2. Oftentimes it was pointed out that the concrete in the compression zone behaves in much the same way as axially loaded compression specimens. There is certainly no assurance of this, since it is not true for static tests and of course there is a strain gradient in the beam which does not exist in compression specimens. No fatigue compression failures were noted in any of the beams reported except those of Le Camus.

3. On occasion, beams failed in diagonal tension fatigue but the real cause of failure was obscured by bond and shear combination failures. Tests have been reported in which beams have failed in shear in repeated loads as low as 40 percent of the ultimate strength. Data is scarce on this phase.

4. Beams accumulate residual deflections under extensive fatigue loading much the same as plain concrete specimens but recover somewhat during rest periods.

Fatigue of prestressed concrete

The use of prestressed concrete in structures of all kinds has become popular in recent years. It is natural that engineers would question the use of this product when first confronted with the higher concrete stresses and the extremely high bond stresses which exist. From a fatigue standpoint the methods of failure are essentially the same as for a conventionally reinforced beam, i.e., (1) fatigue of concrete in the compression zone, or in diagonal tension, (2) in the prestressing steel, (3) in bond, and (4) for post-tensioned beams by the fatigue failure of anchorages and splices. The fatigue of prestressed concrete differs in one important aspect—in working load range the variations in stress are small but vary about some high mean stress in both the constituent materials.

Typically, when first using prestressed concrete, the engineers demanded acceptance testing in many cases to assure the buyer that he was obtaining a product as good as, or better than, reinforced concrete. Unfortunately, both the early static and fatigue tests carried on for these purposes were inadequately instrumented, or a proper, suitable criterion for performance was not established. Consequently, early conclusions are limited to the performance of a particular beam under the testing conditions used. There are numerous such tests in the literature, some extensively described, some only briefly mentioned. The bibliography lists several of these publications [65-88] but all of them cannot be described because of space limitations. The beam cross sections are shown in Fig. 10. A recent summary of fatigue investigations through 1955 has been summarized by Rowe.[4]

Apparently the first tests on prestressed concrete were carried out by Freyssinet[66] in 1934. He compared the performance of two hollow telegraph poles in fatigue—one reinforced concrete and the other prestressed. The prestressed pole was in good condition at the end of the test but the conventional pole had failed.

Another early use of prestressed concrete in Europe was for railroad sleepers (ties). Abeles[68] examined the performance of ties which had been in service over 2 years from which he concluded that there was no substantial difference between the new or used ties. The used ties had been loaded an estimated 4,000,000 times; nevertheless, the bond was unaffected and in static tests failure was by yielding or fracture of the steel. Campus,[69] too, investigated the performance of ties of the Hoyer and Atlas types. One of the Hoyer types was intact after 10,000,000 cycles of load but three failed by slip of the wire accompanied by shear failure at 5,000,000 to 6,000,000 cycles. Two others failed by combined shear and bending at over 9,000,000 cycles. The service conditions of ties and also these tests cannot be considered typical of prestressed concrete structures, however, because of their shortness and high shears.

Typical also of early tests, Lebelle[75] tested three rectangular beams prestressed with a number of 0.2-in. diameter wires. Ciment Fondu was used in the concrete (cube strength 10,000 psi) for two beams, while portland cement was used for the other (cube strength, 4,670 psi). One of the beams

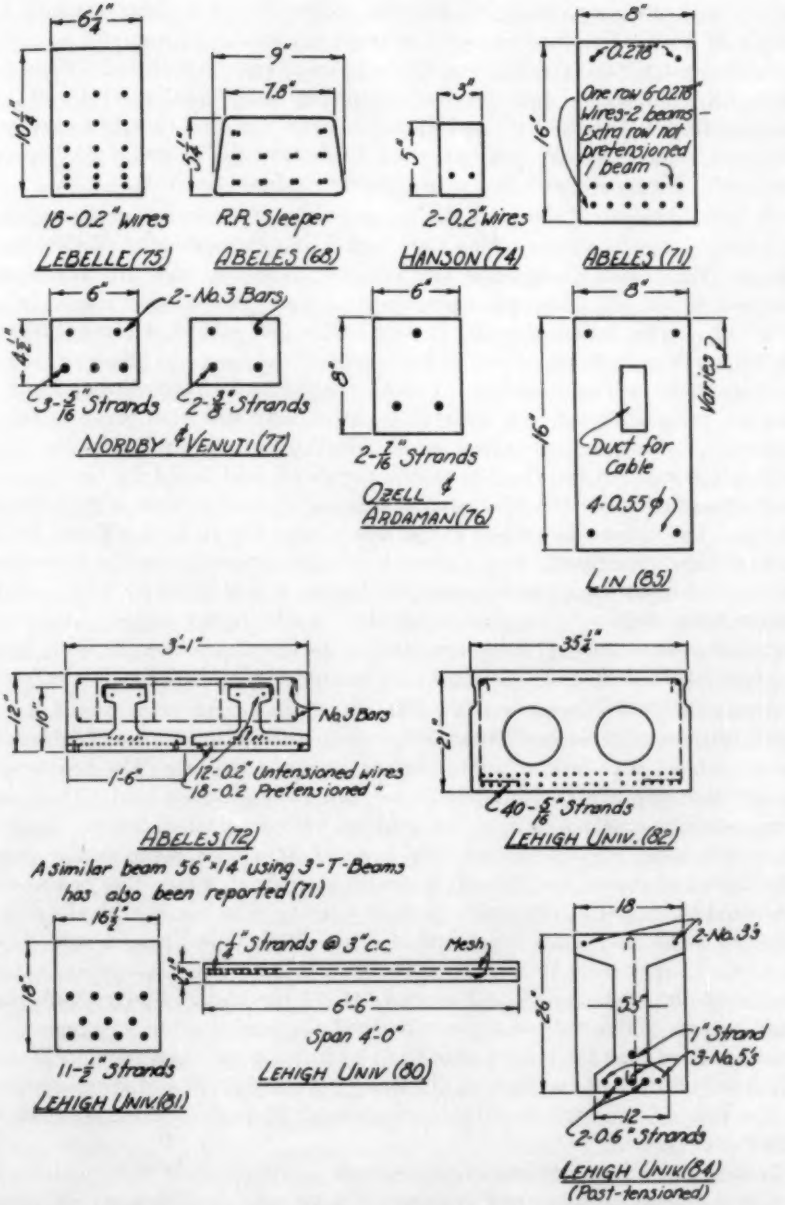

Fig. 10—Cross sections of some of the prestressed concrete beams tested under repetitive loading

of the first category endured 1,400,000 cycles of load ranging between 1.03 and 1.54 of the live load moment without cracking. It was still in perfect condition after 300,000 additional cycles of load between 0.46 and 1.6 live load moment. The second beam was unaffected by a half million cycles of loads ranging from 0.5 to 2.0 live load moment. The portland cement beam failed in shear after 210,000 cycles alternations between 0.75 and 1.25 live load moment. No general conclusions can be drawn from the tests.

In 1951 Abeles[70-72] reported the fatigue tests of three composite partially prestressed concrete slabs which were tested in collaboration with Campus at Liege. The beams consisted of two inverted T-beams, each prestressed with eighteen 0.2-in. diameter wires per beam with an integral slab placed in situ (Fig. 10). The beams were 20 ft long. The first slab was given 1,000,000 cycles so that the lower fiber had a hypothetical range of stress of 550 psi tension to 50 psi compression. Cracks opened and closed during the tests but no permanent set was noticed. The second slab was given 3,000,000 repetitions of a progressive load applied in three stages such that the hypothetical tension for the third million was 950 psi and failed by fracture of a wire in a gage slot. The third slab also failed by steel fatigue after 2,500,000 cycles. The lower fiber stress range was minus 100 to plus 650 psi for the first million repetitions, and although cracks appeared during the second million the same load was continued to failure at 2,500,000 by fatigue of the prestressing steel. Abeles concluded that cracks under service loads were undesirable because steel fatigue seemed to be directly related to them but if fracture does not occur by fatigue the ultimate carrying capacity is unaffected.

Although bond failures were rare in all of the fatigue tests reported, two investigations of prestressed beams report such failures. Hansen[74] tested ten beams subjected to fatigue, vibrations, or repeated impact. The beams were loaded from zero to approximately 75 percent of the ultimate load. The beams were severely cracked during the loading. When clean 0.208-in. diameter wires were used, they slipped at 7200 and 654,000 cycles, while similar beams with rusted wires endured 170,000 cycles before failure. Four other beams were subjected to impact by the drop of a 32-lb weight. The beams with clean wires failed in bond after only five drops of 7 in. The typical rusted wires beam took over twenty 10-in. drops. It was evident that the bond failure mechanism started at the center crack and moved toward the ends. Failure took place when a peak of flexural bond stress reached the prestress transfer region. It must be noted that the beams were 72 to 84 in. long and thus were quite short which may have had an effect on the bond. No serious loss of stress occurred in the beams subjected to vibratory loads at 22 percent ultimate load for 10,000,000 cycles.

In the United States the use of steel strands ranging from $\frac{1}{4}$ to $\frac{1}{2}$ in. diameter has been popular. Ozell and Ardaman[76] have reported tests on six beams 19 ft long loaded with a center loading. One beam was loaded for 1,000,000 cycles at less than the cracking load without ill effect and even when the load was increased to slightly beyond the cracking load the beam withstood

2,000,000 additional cycles. Other beams were subjected to repetitive loads ranging from 180 percent to nearly 300 percent of the design loads. In all of these cases the beams failed by fatigue of the steel strand. The number of cycles at failure ranged from 126,000 to 940,000 depending on the range of stress induced in the strand (18,000 to 80,000 psi). The minimum stress was 142,000 psi. No bond failures occurred other than local failures in the immediate crack zone. It was concluded that the use of 7/16-in. diameter strands seemed feasible and that the flexural fatigue limit of the beams tested was about 1.8 times the design load.

Both lightweight aggregate concrete (Idealite) and conventional concrete were used in similar tests of beams using 5/16- and ⅜-in. strands by Nordby and Venuti.[77] The beams were matched in pairs, and one of the pair was subjected to a fatigue load ranging from 30 to 70 percent of the ultimate load for various numbers of cycles of load. The two beams were then compared by static loading to failure. The static strength of the fatigued beams was not impaired by one or two million cycles of the design load even when severely cracked. In one case a conventional concrete beam endured 10,000,000 cycles of the design load (29 percent ultimate) without injury. Three fatigue failures were received in the steel strand at 136,000 to 842,000 cycles with a range of stress of approximately 24,000 psi (minimum stress 150,000). There was no difference in the fatigue performance of either concrete used. No bond failures were received due to fatigue; in fact, in beams statically preloaded so that slight slip of strand had occurred, additional fatigue cycles did not cause additional damage. In general, it seemed that strand was superior to smooth wire because its spiral shape gave high mechanical bond. It was concluded that formation of cracks formed stress discontinuities which caused steel fatigue failures if the range of stress was great enough.

Other tests have been performed on variety of beams at Lehigh University by Ekberg, Eney, et al.[78,80-83] Most of these beams were full-scale highway or railway bridge beams prestressed with steel strand ranging up to ½ in. in diameter. In one case a 38-ft highway bridge beam containing forty 5/16-in. strands was subjected to H-20 loading for 1,300,000 cycles and 100,000 repetitions of 54 percent overload without ill effect. Details of all the Lehigh tests have not been published.

Five investigations performed abroad were concerned with post-tensioned concrete beams. The first was by Magnel[85] who tested two narrow T-beams over 35 ft long and 3 ft deep. The beams were humped-back with a rise of 13 in. and were prestressed with a 24-(0.2-in. diameter) wire grouted straight cable. The second beam was identical except that indented wire was used. Several wires failed under a repetitive load gradually increased after each million cycles which was terminated at over 6,000,000 cycles. The second beam failed by steel fatigue (nine wires) at 4,800,000 cycles of load similar to the first beam. The estimated stress in the wires was 60-65 percent of the ultimate stress. The safety factor against dynamic failure was approximately 2 for both beams while that against dynamic cracking was 1.51 as compared

with 2.5 for static loading. He concluded that removal of grips after grouting was a sound procedure. Billington[86] performed tests on a beam similar to Magnel's except a special bond anchorage was used at each end and the cable was not grouted. The dynamic safety factor was slightly less than Magnel's beams (1.4) and failure by steel fatigue occurred sooner at 4,680,000 cycles where seven wires fractured. The estimated steel stress was 81 percent of the ultimate stress at failure. The dynamic safety factor against failure was possibly somewhat greater than Magnel's beams because of the lack of stress concentrations at cracks (beams were not grouted). The wire spacing grillages were badly damaged as a result of the tests. Xercavins[88] performed tests similar to Magnel's except for the use of a special steel with a higher fatigue strength. Steel fatigue was the only cause of failure although the safety factors were higher than Magnel's because of the special wire used. It was evident that the various steels should each be evaluated for their potential fatigue resistance. Lehigh University[83] has also reported on a post-tensioned member (see Fig. 10).

Lin[87] has tested the only continuous prestressed beams. The two beams were two-span beams of over 50 ft over-all length and post-tensioned with thirty-two 0.2-in. wires in a grouted duct of varying depth in the beam. Fatigue failures of the wires took place at the grillage separators under repeated heavy loads causing an average stress 25 percent higher than the prestress. The beams failed at 3,200,000 and 4,400,000 cycles, respectively. The factors of safety against rupture under fatigue were 2.6 and 2.2.

Summary of prestressed concrete results

Again, these statements should not be regarded as definite conclusions.

1. In none of the tests did concrete fail by fatigue. The current working stresses seem to give adequate protection in this regard.

2. Fatigue failure of stressing wires or strands was the cause of all failures reported. These failures seemed to be related to the extent and severity of the cracks.

3. Bond failures were rare and were found only under unusual circumstances, i.e., short beams, short shear span.

4. The ultimate strength of prestressed beams for static loads was unaffected by repetitive loading if they did not fail by fatigue.

5. Safety factors seemed to be approximately 2 against fatigue failure for most of the beams tested.

6. Prestressed beams seemed superior to conventional beams for resisting fatigue loading. In fact, in a recent paper, Ekberg and Walther[79] analytically verified this by relating the modified Goodman diagram of both the concrete and prestressing steel to the theoretical stresses in both types of beam.

7. Further research is needed on bond failure and the action around cracks. Little progress can be made in this direction until these phenomena are understood for static loads. Efforts should be made toward establishing modified Goodman diagrams for both high strength concrete and steel as an aid to the analysis of prestressed beams subjected to fatigue loading.

CONCLUSIONS

Most of the research up to this time has been exploratory and investigators now know what to look for in their experiments. More research is needed in every phase of fatigue of concrete and future investigations should be well organized to isolate a particular variable. Research on the fundamental properties of concrete fatigue to describe the mechanism of fatigue failures may be particularly fruitful. An understanding of this mechanism would make previous tests more valuable as well as improving those made in the future.

More research is needed to investigate the effect of moisture, aggregates, aggregate bond, curing, rest periods, microcracks in the paste, different environments of corrosive agents, specimen size, range of stress, combined loading, freezing and thawing, air entrainment, admixtures, temperature cycles, moisture cycles, accumulative fatigue damage, and previous stress histories on fatigue of both plain and reinforced concrete. In reinforced concrete efforts must be toward understanding the mechanism of bond and the mechanism of failure around tension cracks. A solution to this problem may be found in the newer x-ray methods pioneered by Evans.[89] Other work must relate the results of fundamental properties to reinforced concrete structures.

The potential economic return for evaluation of these problems is almost fantastic. The saving in highway construction alone would be enormous if the life of concrete pavements could be prolonged 10 years by an understanding of fatigue. But greater funding for fundamental research will be necessary from both state and federal governments as well as industry to accomplish the task. Industry especially must modify its viewpoint to consider such investigations as a long-time investment which will pay dividends in increased use of concrete over the years.

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Effect of Range of Stress on Fatigue Strength of Plain Concrete Beams*

By JOHN W. MURDOCK† and CLYDE E. KESLER‡

SYNOPSIS

From the results of tests of 175 plain concrete beams subjected to repeated flexural loading, it was found that plain concrete exhibits no fatigue limit when subjected to loads which produce no reversals of stress. Fatigue strengths at ten million repetitions of stress were determined for each of the several ranges of stress investigated, and these strengths were found to be dependent on the range of stress to which the specimens were subjected.

The test results are in good agreement with previous investigations insofar as valid comparisons may be made. The need for additional research is, however, most apparent; the results obtained from this investigation do not permit the definition of the fatigue behavior of plain concrete subjected to reversals of stress, nor do they permit a valid description of the process by which fatigue failures occur.

INTRODUCTION

The increased application of concrete as an engineering material demands an additional knowledge of its behavior under the several types of loading to which it is subjected. Such knowledge is necessary not only to provide safe, efficient, and economical designs for the present, but to serve as a rational basis for the conception of improved and extended applications for the future.

The behavior of plain concrete when subjected to repeated or fatigue loading can only be approximated. The existence of a fatigue limit has not been established, nor have fatigue strengths been defined for any finite life of the material. The effects of service-encountered variables which may alter the behavior may only be inferred, and the mechanism of failure is not clearly defined.

The controlled laboratory investigation affords an effective tool for the determination of this information.

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Scope

The discussion of the investigation reported in this paper will be confined essentially to the following points:

1. The establishment of a fatigue limit for repeated loading, or, if no such limit exists, the establishment of fatigue strengths for probable finite lives.
2. The effect of the range of stress on the behavior of plain concrete beams subjected to repeated flexural loading.

PREVIOUS INVESTIGATIONS

Although the previous investigations which have been made to determine the fatigue behavior of plain concrete are relatively few and limited in extent, they do indicate a pattern of behavior to which the results of more recent investigations may be compared.

Repeated load tests made on tension briquets by DeJoly and reviewed by Mills and Dawson,[1] together with repeated load tests on compression specimens by Van Ornum,[2,3] indicate the possible existence of a fatigue limit of the material.

Repeated load tests on compression specimens by Probst and Mehmel as reported by Moore and Kommers[4] indicate a fatigue limit for repeated compressive loading which lies between 47 and 60 percent of the static ultimate compressive strength. In addition they indicate a change in the fatigue limit with an alteration of the range of stress.

Investigations which indicate the behavior of plain concrete subjected to repeated flexural loading have been reported by Clemmer,[5] Hatt and Crepps,[6,7,8] Williams,[9] and Kesler.[10] In brief, Clemmer determined a fatigue limit for plain concrete subjected to loads which produced stresses ranging from zero to maximum in tension. This limit was approximately 55 percent of the static ultimate flexural strength. Hatt and Crepps determined the same limit for beams in which the loads produced complete reversals of stress.

In Williams' tests of lightweight aggregate beams no fatigue limit was found, but fatigue strengths at one million repetitions of stress were determined both for specimens subjected to stresses ranging from zero to a maximum in tension and for those subjected to complete stress reversal. The limits were somewhat lower than those found by Clemmer, Hatt, and Crepps.

Kesler found no fatigue limit but established fatigue strengths at ten million repetitions of stress ranging from a small value in tension to some maximum value in tension. The fatigue limits determined were approximately 62 percent of the static ultimate flexural strength and, with negligible differences, were independent of the speed of testing, which ranged from 70 to 440 cycles per minute.

Although there appear to be certain inconsistencies among the results of the several investigations, they do afford a basis for further tests and provide a guide for the interpretation of additional data. In order that the information may be used effectively it is necessary that the apparent inconsistencies be reconciled or that erroneous information be corrected.

SPECIMENS AND TEST PROCEDURE

Type I portland cement was used in all specimens and the aggregates were well graded Wabash River sand and gravel which passed the usual specification tests. The coarse aggregate had a maximum nominal size of 1 in., and the fineness modulus of the sand was approximately 3.0. Each of the mixes was proportioned by trial batching to yield concrete with an ultimate compressive strength of approximately 4500 psi. Mixes of dry consistency had average slumps of 1 in.; those of wet consistency had average slumps of 6 in.

One test specimen and four control cylinders or two control beams were cast from a single batch and the concrete was compacted with mechanical vibrators. Test specimens, cylinders, and control beams remained in the forms for 24 hr; they were then moist cured for a period of 6 days and removed to storage in normal laboratory atmosphere until tested.

All test specimens were plain concrete beams 6 x 6 in. in cross section and 64 in. long. The companion specimens were either standard 6 x 12-in. control cylinders or control beams 6 x 6 x 21 in.

Loading

All specimens were simply supported on a 60-in. span and subjected to repeated loads of varying range and magnitude applied at the third points. The central third of the test specimen was thus subjected to pure bending, the shearing stresses due to the weight of the specimen being negligible.

The range of the repeated loads was systematically varied throughout the investigation. The variation within each series, with one exception, was such as to maintain a constant ratio R between the minimum and maximum applied stresses.

Series of tests were made with concrete of both wet and dry consistency in which R had values of 0.25, 0.50, and 0.75. One series of tests was made in which the minimum applied load and stress was constant, R then being a variable throughout the series. This modification was made because the manner in which the loads were applied precluded the reduction of the minimum applied load to zero without the simultaneous introduction of impact loads. An arbitrary minimum load of 100 lb was used throughout this series.

Equipment

Three testing machines were used in this investigation, one of which is shown schematically in Fig. 1. The specimens were supported and loaded through a system of rollers and balls to prevent the accidental introduction of torsional forces. The repeated loads were applied by a loading lever through an

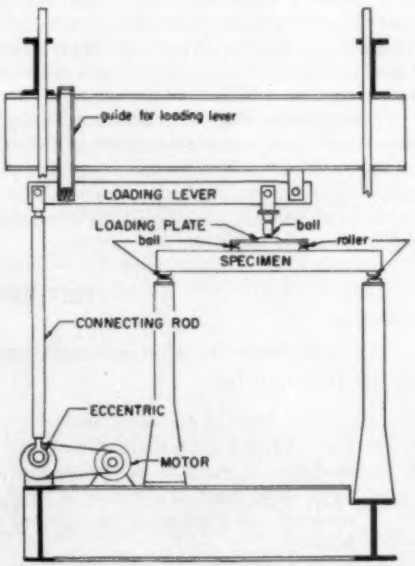

Fig. 1—Schematic diagram of fatigue testing machine

adjustable loading bar and loading plate. The lever was actuated by a connecting rod attached to a motor driven variable eccentric.

The range of load was determined by the set in the variable eccentric while the intensity of the load was controlled by the adjustable loading bar and measured by the loading plate which served as a dynamometer. Necessary correction or readjustment of the load during the course of the test was accomplished by varying the length of the connecting rod.

The machines operated continuously, applying between 400 and 440 repetitions of load per min. In all tests the machines were stopped automatically whenever a specimen failed. The repetitions of load applied to each specimen were recorded by a counter.

Estimation of ultimate flexural strength

An accurate estimate of the ultimate flexural strength of the test specimen was a desirable prerequisite for the selection of a fatigue load; for interpretation of the test results it was essential. The inability to duplicate concrete specimens invalidates the measurement of fatigue resistance solely in terms of the intensity of the applied stress; a more satisfactory measure is made in terms of the static ultimate flexural strength. Once a specimen fails under repeated loading it is obviously impossible to determine the static strength at the point of failure; however, a series of statistical studies by Kesler[11] affords a means of approximating this strength.

For specimens cast with control cylinders the initial estimate of the flexural strength was made from the statistical relationship between cylinder strength and beam strength. Although statistically sound, when applied to single specimens these estimates were frequently quite erroneous and loads unreasonably high or low were then applied in the tests. In an attempt to reduce the error, companion flexure specimens were substituted for control cylinders. The average flexural strength of the two companion specimens was taken as the estimate of the flexural strength of the test specimen; this procedure reduced, but did not eliminate, the error.

The statistical study also in licated a one-to-one correlation between the modulus of rupture of a specimen tested on a 60-in. span and the same property determined from tests of the broken portions of the specimen tested on an 18-in. span when both types of specimens were subjected to third-point loading. The "true" flexural strength of the test specimens was therefore assumed to be the strength determined from the broken portions of the specimen upon completion of the fatigue test.

For specimens with which no control cylinders were cast, the quality of the concrete was determined from compression tests of the modified cubes obtained from the flexural tests of the specimens. The correlation with cylinder strength was again based on previously mentioned statistical studies. It is considered that the error which may have been introduced by the application of statistical correlation was offset by the use of the actual specimen in determining the quality of the concrete.

TEST RESULTS

Notation

The following notation is used throughout the presentation and analysis of the test results:

f_1 = flexural stress at minimum load
f_2 = flexural stress at maximum load
f_r = modulus of rupture as determined from static tests of portions of test specimen after failure under repeated loading
f_c' = ultimate compressive strength as determined from standard 6 x 12-in. control cylinders
$R = f_1/f_2$ = a measure of the range of loading
$F = f_2/f_r$ = fatigue factor
$M = f_1/f_r$ = a measure of the minimum stress
F_{10} = F at ten million repetitions of stress = fatigue strength

All stresses are computed by the usual flexure formula and so represent the theoretical rather than the actual intensities of stress which existed in the specimen.

Results

Concrete of both dry and wet consistency was used in these tests. Whenever possible, specimens which had aged a minimum of 90 days were used so that the concrete would not continue to gain strength during the course of the test and so influence the results.

In Fig. 2 to 5 are plotted the data obtained from the four series of tests.* These data have been plotted in a manner similar to a conventional *S-N* diagram, the ordinates to the curve being the fatigue factors F, and the abscissas the logarithms of the cycles or repetitions of stress sustained prior to failure.

In none of the test series did the data indicate the existence of an endurance limit, although many of the specimens sustained more than ten million repetitions of stress without failure.

It was concluded that a straight line would best represent the curve determined by each set of data, and these lines were established by the method of least squares. In making the computations the cycles to failure were assumed to be the independent variable. The straight lines would, of course, have different slopes were the fatigue factors assumed as the independent variables; however, since there is a greater probability of error in the fatigue factors it was felt that the method employed was the more likely to give a proper analysis of the data. The following groups of specimens were excluded from the computations:

1. Specimens which did not fail during the course of the test. No accurate estimate could be made of the true position of these specimens on the plot.

2. Specimens which failed at less than 50 repetitions of stress. It was concluded that many of these failures may have been influenced by other variables and were not representative fatigue failures.

3. Specimens which were retested under loads of increased intensity. The degree to which the behavior of these specimens was influenced by the previous loading history could not be accurately assessed.

Notations of these exclusions are made on the figures.

Test Series I

The minimum stress f_1 was constant for all specimens and equal to 70 psi. The ratio R ranged between 0.13 and 0.18.

It will be noted that several of the specimens used in Series I had aged only 40 to 64 days, but, with one exception, the tests in which these specimens were used were completed in less than a day. The test results could not have been influenced by an increase in concrete strength.

*Complete tabular data are available from ACI headquarters on request.

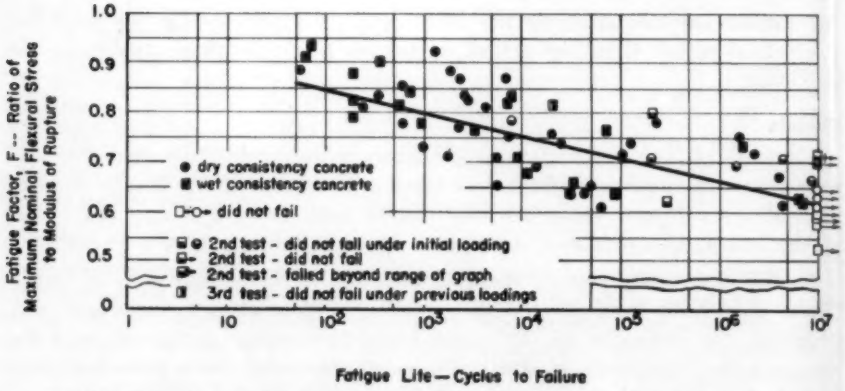

Fig. 2—Behavior of plain concrete under fatigue loading, $R = 0.13$ to 0.18

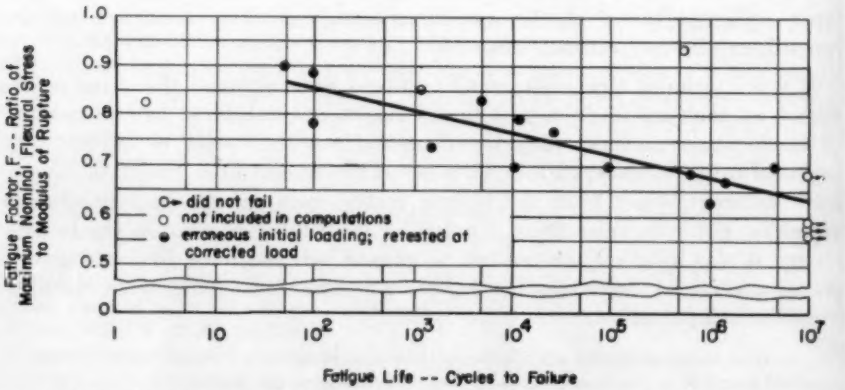

Fig. 3—Behavior of plain concrete under fatigue loading, $R = 0.25$

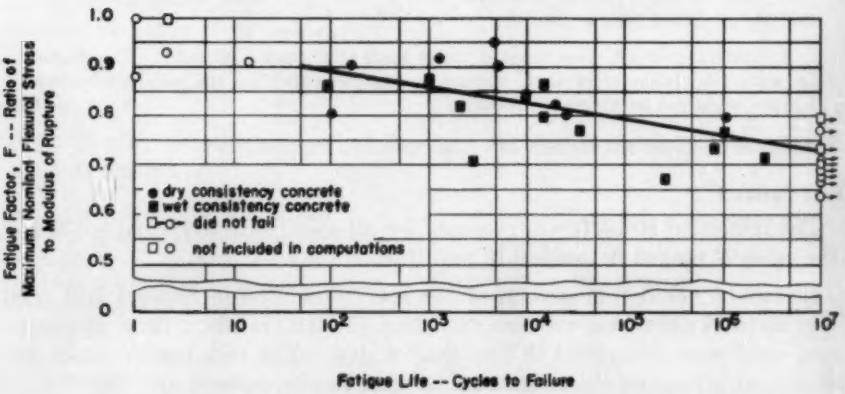

Fig. 4—Behavior of plain concrete under fatigue loading, $R = 0.50$

A fatigue strength, at ten million repetitions of stress, of 61 percent of the static ultimate strength is indicated in Fig. 2.

Test Series II

In all tests in this series, R was constant and equal to 0.25. The range of loading was thus decreased somewhat from that in Series I.

The fatigue strength, at ten million repetitions of stress is 63 percent of the static ultimate strength (Fig. 3), only slightly higher than the strength determined from the tests of Series I.

Test Series III

In this series of tests, R was held constant and equal to 0.50, a further reduction in the range of loading.

A fatigue strength, at ten million repetitions of stress, of 73 percent of the static ultimate strength is indicated in the plot in Fig. 4. This limit is significantly greater than either of those which was determined in Series I or II.

Test Series IV

For all specimens in this series R was held constant at 0.75, the minimum range of loading for any of the series of tests.

It will be observed that the scatter of the data in this series of tests seems somewhat more pronounced than that observed in either Series II or III. This may be due, in part at least, to the fact that all of the specimens were tested at a rather high percentage of their static ultimate strength; no specimen failed at less than 73 percent of the static ultimate strength. The curve, however, seems to be a reasonable statistical average.

Fig. 5 indicates a fatigue strength, again at ten million repetitions of stress, of 85 percent of the static ultimate strength, a significant increase with a further reduction in the range of stress.

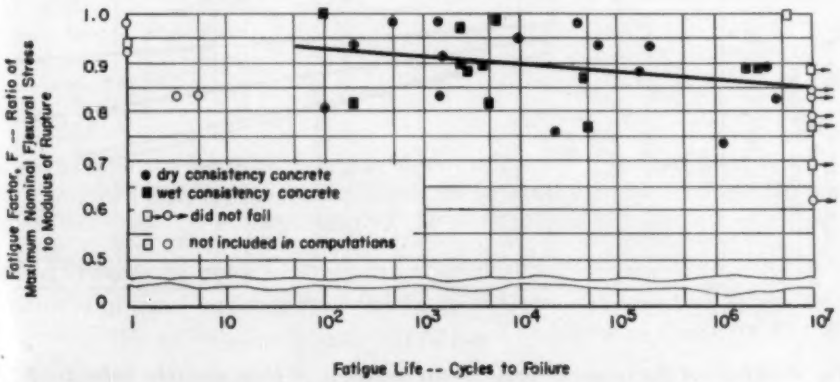

Fig. 5—Behavior of plain concrete under fatigue loading, $R = 0.75$

Comparison of test results

The four curves are shown for comparison purposes in Fig. 6. The influence of the range of stress, as measured by the ratio R, may be seen; successive fatigue strengths, for R equal to 0.75, 0.50, 0.25, and 0.13 to 0.18, are 85, 73, 63, and 61 percent, respectively, of the static ultimate strength. The difference between successive fatigue strengths is constantly diminishing, suggesting the possibility of a lower limiting value.

Although concretes of both wet and dry consistency are consolidated in these series, the results are not influenced thereby. Separate computations were made for both series and the differences were negligible.

The value of R is necessarily approximate in Series I. Considering all specimens in the series, the average value of M was 0.11, so as F varied from 0.86 to 0.61, R ranged between the approximate limits of 0.13 to 0.18.

In Fig. 7 the effect of the range of stress on the fatigue limit is shown by the use of a modified Goodman diagram. In this diagram the ordinates to the 45-deg line passing through the origin represent the values of M, the ratio of the minimum stress to the modulus of rupture. Ordinates to the curve at the same abscissas, represent the fatigue strength F_{10}, taken at ten million repetitions of stress, which may be obtained with the corresponding value of M. As the minimum stress is made a greater percentage of the ultimate strength, greater fatigue strengths may be obtained for specimens in which no reversals of stress occur.

The data from the tests by Hatt and Crepps afford the only approximation of the fatigue strength when concrete is subjected to reversals of stress. However, the tests summarized at the beginning of the paper indicate that the fatigue limit for concrete subjected to repeated compressive stresses is approximately 50 percent of f_c'; compressive flexural stresses are but a small percentage of this limit when the flexural tensile stresses reach the critical

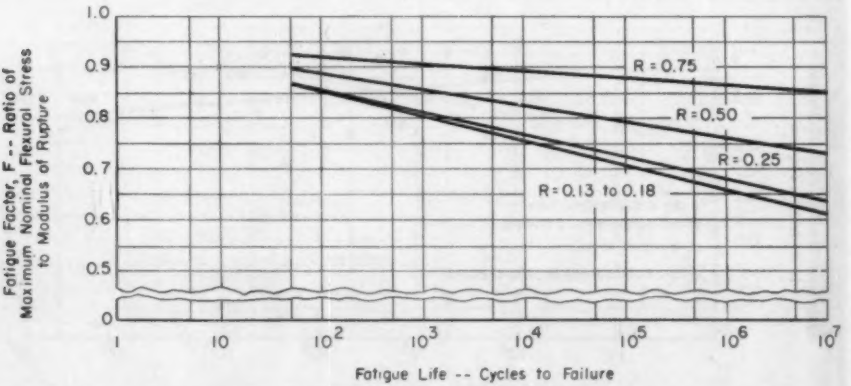

Fig. 6—Effect of the range of stress on the behavior of plain concrete under fatigue loading

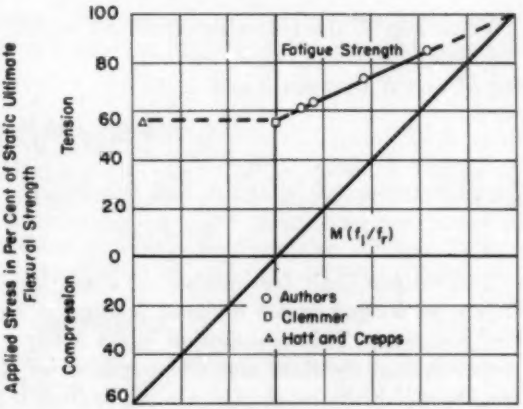

Fig. 7—Modified Goodman diagram showing the effect of the range of stress on the fatigue limit of plain concrete under repeated loading

values described in this paper. It seems reasonable to assume that tensile stresses remain critical as the stresses are reversed and the fatigue strength is therefore approximated as remaining constant at 56 percent of the static ultimate flexural strength.

The tests by Clemmer and Hatt and Crepps are included in Fig. 7 to show the correlation among the Illinois and Purdue tests and those reported in this paper. Dotted portions of the curve represent regions in which there is limited substantiating data.

ANALYSIS AND CONCLUSIONS

Fatigue strengths

Fatigue strengths, at ten million repetitions of stress, have been determined as follows:

Ratio of minimum to maximum stress	Fatigue strength as a percentage of f_r
0.75	85
0.50	73
0.25	63
0.00	56*
−1.00	56*

In none of the series of tests was there evidence of a fatigue limit; it must be concluded that no fatigue limit may be assigned for plain concrete of the type tested and within the range of the tests reported in this investigation.

Effect of range of stress

In the series of tests reported the range of stress was found to have a significant influence on the fatigue strength. This effect is shown in Fig. 6 and 7.

*Extrapolated from Fig. 7.

From Fig. 7, the following equations, which may be used to approximate the fatigue strength, were derived:
for M and R between 0 and 1,

$$F_{10} = 0.56 + 0.44\,M, \text{ or}$$
$$F_{10} = 1.3/(2.3 - R)$$

for M between -0.56 and 0, and R between -1 and 0

$$F_{10} = 0.56$$

The fatigue limit for reversal of stress is based on the assumption that failure in fatigue under reversed loading is controlled by tensile failure in the concrete. This assumption gains some measure of support from the tests reported by Hatt and Crepps; however, there is a need for additional tests in which the range of stress varies from a partial reversal to a complete reversal.

Limitations of the test procedure

The accuracy with which the fatigue strengths have been established is determined by the accuracy with which the ultimate static flexural strength of the specimens was approximated. In a few of the static tests there were significant differences between the indicated flexural strengths of the two broken portions of the test specimen, and the average of these strengths may not have been indicative of the true flexural capacity of the unbroken specimen. The modulus of rupture, even when determined from static tests which were in close agreement, may have been somewhat different from that of the original specimen. These limitations must have influenced the amount of scatter in the data.

Despite these limitations, the trend of the data in each of the tests is well defined, and it is considered that the curves established from these data are representative of the fatigue behavior.

CONCLUSIONS

The data obtained from this investigation give support to the following conclusions:

1. Plain concrete, made with sand and gravel aggregates, subjected to repeated flexural loading, exhibits no fatigue limit, at least through ten million repetitions of stress.

2. The repeated loads which plain concrete may sustain for a finite number of repetitions without failure is a critical percentage of the static ultimate flexural strength. Furthermore, this percentage is a function of the range of stress to which the concrete is subjected.

3. The data obtained in this investigation, taken in conjunction with that of the Illinois and Purdue tests, tentatively define the behavior of plain concrete subjected to loadings which produce reversals of stress. Although the agreement among the investigations is generally quite good, it cannot

be construed as conclusive. Additional research is therefore needed to define the fatigue behavior in the region where the concrete is subjected to reversals of stress, both partial and complete.

ACKNOWLEDGMENT

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Title No. 55-13

From a symposium sponsored by ACI Committee 215

Probability of Fatigue Failure of Plain Concrete*

By JOHN T. McCALL†

SYNOPSIS

Fatigue tests were run on concrete beams and the data were analyzed in an attempt to determine the relationship for concrete between stress S, number of cycles to failure N, and probability of failure P. It was found that this relationship can be expressed reasonably well graphically; one of the mathematical relationships investigated fitted the data at high stress levels fairly well, but the fit at the lowest stress level was poor. The extreme value functions investigated could not be used to describe the relationship.

INTRODUCTION

Fatigue refers to the phenomenon of rupture of a material—when subjected to repeated loadings—at a stress substantially less than the ultimate static strength. Materials exhibit fatigue failure when stressed by bending, direct tension or compression, torsion, or combinations of these.

A fatigue loading on a specimen, or structural part, may be thought of as a load which varies uniformly (as a sine wave) superposed on a static load. The number of cycles to failure is a function of both the static load and the varying superposed load.[1] It appears that the most severe fatigue loading is one in which the mean static load is zero and the varying load causes alternate tension and compression; this is called a completely reversed loading.

GENERAL INFORMATION

Fatigue testing and analysis of data

A fatigue testing program consists of applying repeated loads of different magnitudes, or stress levels, to the specimen and observing the number of cycles of load application necessary for fracture at each stress level.

Until recently it has been almost universal practice to show fatigue test results by plotting the test data in an S-N coordinate system (S = testing stress level, N = number of cycles for failure). The upper curve shown in Fig. 1 is typical of fatigue tests of ferrous metals. The ordinate to the horizontal asymptote of this curve is the stress level at which the specimen will withstand an infinite number of cycles and is called the fatigue limit. Fig. 1

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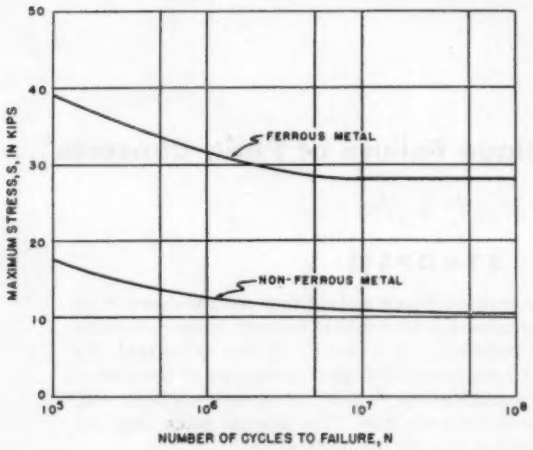

Fig. 1—Typical *S-N* curves

also shows a typical *S-N* curve for nonferrous metals. This curve does not become asymptotic to the horizontal, which means that no fatigue limit exists. For a material having this type of curve it is customary to specify a fatigue strength—the strength of the material for any given life or number of cycles (usually a very large number).

More recently several investigators[2,3,4] have shown that an adequate representation of fatigue data must include the probability of failure P as well as the stress S and the number of cycles to failure N. The fatigue test data shown in Table 1 illustrate this point. As might be expected, the 20 specimens tested at a given stress level failed at different numbers of cycles. If, as is usually true, these data are to be used to predict the future performance of this material, the data are analyzed by ranking the specimens in the order of the number of cycles to failure and calculating the probability of failure P, by dividing the rank of each specimen m by $(n + 1)$, where n equals the total number of specimens tested at this stress level. The calculated P-values are shown in Table 1.

In calculating P, the reason for dividing by $(n + 1)$, rather than by n, is to avoid obtaining a probability of failure of 1.000 for the specimen having the greatest N-value. The probability that all future specimens will fail at this same number of cycles is not 1.000, however close it may approach that limit. The ratio $m/(n + 1)$ appears to give the best estimate of the probability of failure; it was proposed by Weibull[5] and by Gumbel[6] and has been used by Freudenthal[2] and by Sinclair and Dolan.[7]

Concrete fatigue

Although relatively few investigations have been conducted on the fatigue of concrete (see Reference 8 for a review of many of these), it has been shown that fatigue of concrete is similar to that of metals. Important conclusions of some previous works have indicated that:

TABLE 1—FATIGUE LIFE N OF CONCRETE BEAMS LOADED IN REVERSED BENDING

m, specimen number	N, number of cycles to failure in thousands, for stress level* indicated			Probability of failure $P = \frac{m}{n+1}$
	$R = 0.475$	$R = 0.575$	$R = 0.675$	
1	1	1	0.2†	0.048
2	1	2	0.2	0.095
3	1	3	1†	0.143
4	5	5	1	0.190
5	23	7	2.5	0.238
6	3537	8	4†	0.286
7	3848	10	4	0.333
8	6756	21	6†	0.381
9	20,000+	28	6	0.429
10		520	19	0.476
11		618	23	0.524
12		727	31	0.571
13		1052	58	0.619
14		3246	287	0.667
15		4113	306	0.714
16		14,306	385	0.762
17		20,000+	795	0.810
18			1964	0.857
19			3176	0.905
20			20,000+	0.952
$n = 20$				

*Stress Level $= \frac{S}{S_{rp}} = R$

†The mean P-value is used for specimens failing at the same number of cycles.

1. Concrete has a fatigue limit of about 50 to 55 percent of the ultimate static strength.
2. The fatigue limit is increased if a repeated stress less than the fatigue limit is applied prior to the actual test.
3. For tests in which there is no stress reversal the deformation of the concrete decreases when the repeated load is stopped. If a long enough rest period is allowed, before the repeated load is started again, the strain recovery is almost complete.
4. Richer mixes have higher fatigue limits than leaner ones.
5. The fatigue limit of concrete increases with the age of the concrete.

More recently Kesler[9] has found that a definite fatigue limit for concrete probably does not exist. This conclusion is based on a two-dimensional (i.e., S-N) concept of fatigue.

EXPERIMENTAL PROGRAM

Test specimens

The test specimens were air-entrained concrete beams which measured 3 x 3 x 14½ in. Nine beams were cast at a time from a 1 cu ft batch of concrete.

Natural sand and crushed limestone having a maximum size of ¾ in. were used as the fine and coarse aggregates, respectively. Mix proportions were 1:2.9:3.1, the water-cement ratio was 0.52, and the cement content was 5.6 sacks per cu yd. Type I cement was used and Darex AEA was added at the mixer. The concrete was mixed in a Lancaster mixer and placed in steel molds in two layers; each layer was vibrated 30 sec. The average air content of all mixes was 6.5 percent and the average slump was 4 in.

Static tests

Three beams from each batch of nine were tested statically in a universal testing machine using the fixture shown in Fig. 2. The modulus of rupture was used to calculate the static strength of the beams. The SR-4 strain gages shown were used to measure the strain distribution in the beam. The mean strength of these three beams was used as the static strength of the batch; the remaining beams were fatigue tested at certain percentages of the static strength,

TABLE 2—MODULI OF RUPTURE

Beam number	Modulus of rupture, psi, for concrete of given batch number									
	1	2	3	4	5	6	7	8	9	10
1	—	523	777	783	790	840	673	623	546	653
2	700	729	783	800	770	773	643	553	591	733
3	587	568	723	553	676	653	837	503	721	680
Mean	644	607	761	712	745	755	718	560	619	689

TABLE 3—ANALYSIS OF VARIANCE TABLE FOR MODULI OF RUPTURE OF BEAMS

	Sums of squares	Degrees of freedom	Mean square	F ratio
Between	130,205	9	14,467	$F = \frac{14,467}{7598} = 1.90$
Within	144,366	19	7598	$F_{.05}(9, 19) = 2.42$
Total	274,571	28		

S_{rp}. The mean modulus of rupture of all the beams tested was 682 psi. (Static strength data are given in Table 2.)

Dynamic tests

A Sonntag SF-1-U fatigue testing machine was used to apply completely reversed bending loads to the specimen at a rate of 1800 cycles per minute. Pure bending was applied to the beam with the type of fixture shown in Fig. 2. Beams from each series were tested at stress levels varying from 45 to 70 percent of the modulus of rupture. The beams were tested to failure or to 20 million cycles, whichever occurred first.

Table 1 shows the adjusted results of the dynamic tests. The stress levels which were actually used in testing a series of beams were 45, 50, 55, 60, 65, and 70 percent of the modulus of rupture, one beam being tested at each level. When analyzing the results using these stress levels, apparent anomalies, in the form of S-N curves which crossed, occurred—indicating that the stress levels selected for testing were probably too close together. This difficulty was eliminated by combining the original data into three groups and using the mean stress level of each group in the analysis of the data.

ANALYSIS OF DATA

Static tests analyzed

An analysis of variance of the moduli of rupture of the beams showed no significant difference among the static strengths of the batches at the 95 percent significance level. An F-value of 1.90 was calculated from the data. For the difference to be significant at the 95 percent level, F would have to exceed 2.42. (If two quantities are different at the 95 percent level it means that chances are 95 out of 100 that this difference is real and not due to chance alone; in the long run there will be a real difference 95 percent of the time.) The F-value is a statistic, or number, used to decide whether or not a difference exists at a selected significance level. The analysis of variance data are given in Table 3.

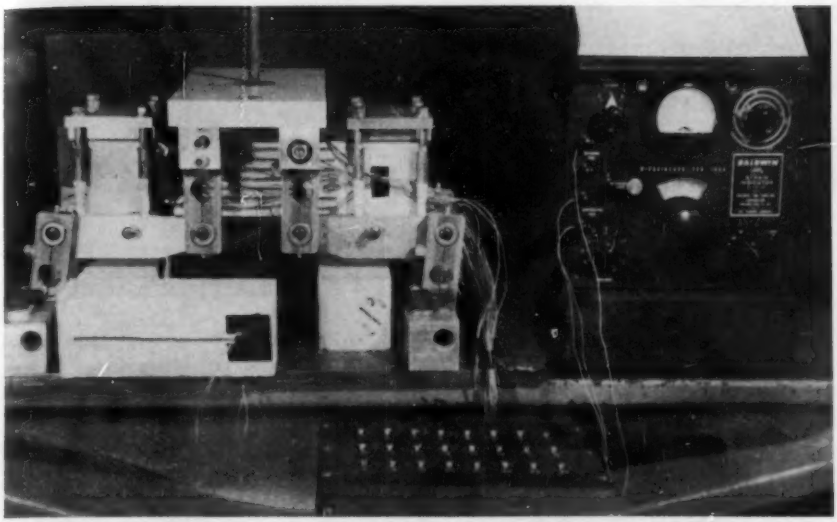

Fig. 2—Concrete beam with SR-4 gages mounted

Dynamic tests analyzed

Family of S-N *curves by graphical analysis*—In this analysis of the data a graphical method similar to that used by Freudenthal[2] and by Sinclair and Dolan[7] was used to construct the family of *S-N* curves. The first step consisted of plotting the data of Table 1 as the probability of failure P against the number of cycles to failure N for each stress level R. ($R = S/S_{rp}$ where S is the stress used in the test and S_{rp} is the mean static strength of the batches.) This plot is shown in the lower left hand part of Fig. 3. From the *N-P* curves, obtained in this manner, a family of *S-P* curves was drawn, and from this the desired family of *S-N* curves was derived. These families of curves are shown in Fig. 3.

In order that each of the final *S-N* curves cover the same range it was necessary to extrapolate one of the *P-N* and all of the *S-P* curves beyond the range of actual observed values. All extrapolated curves are shown by a dotted line.

Mathematical analysis of fatigue data—Another way to describe the *S-N-P* relationship would be by means of an equation or mathematical model. One might intuitively guess at the form of such an equation and then test its applicability by comparing the predicted values of the variables (i.e., values calculated from the equation) with the values obtained from the test data.

A function of the type which might be used to described the *S-N-P* relationship was investigated for concrete fatigue data. An equation of the form,

$$L = 10^{-aR^b(\log N)^c} \quad \ldots\ldots\ldots\

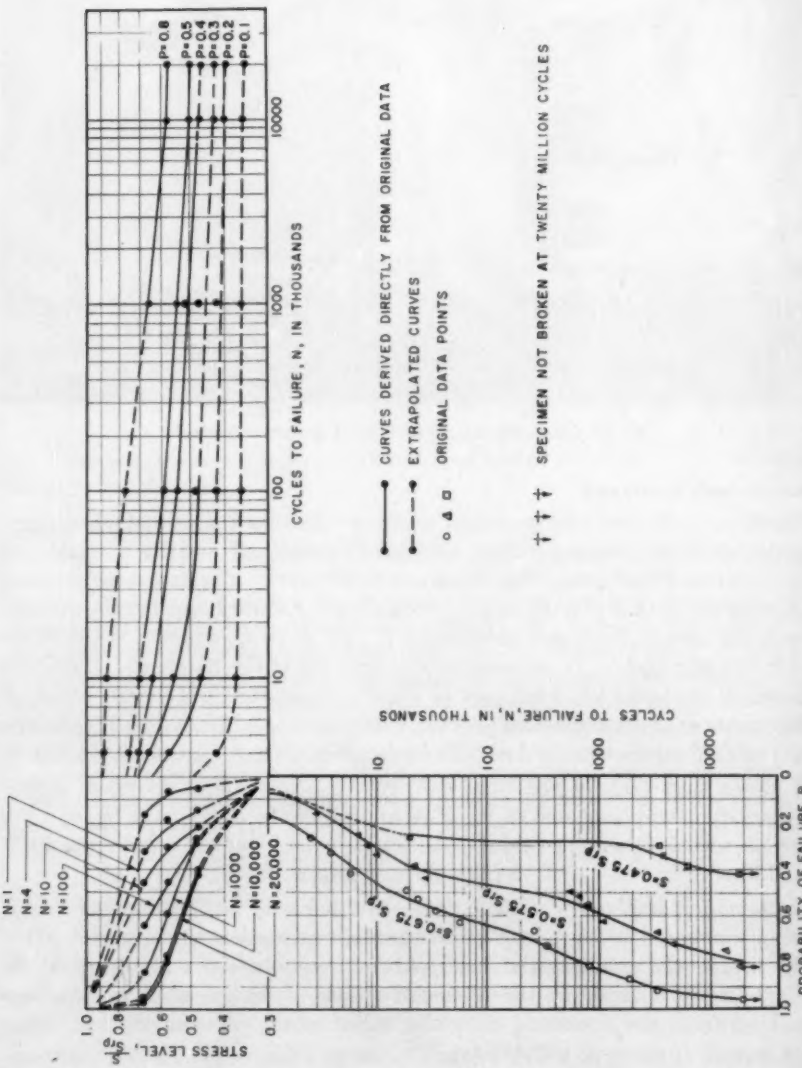

Fig. 3—*S-N-P* diagram for concrete beams in reversed bending

was suggested by the general form of the S-N curve and by relationships used by other investigators.[2,7,10,11] In this equation L is the probability of survival and is therefore equal to unity minus the probability of failure (i.e., $L = 1 - P$). The equation for L is simpler than one for P; this was the only reason for the change of variables from P to L.

That Eq. (1) is valid at limiting values of N and R may be seen by calculating L-values for these limits. For the values of $N = 1$ and $N \rightarrow \infty$ the corresponding values of L are $L = 1$ and $L \rightarrow 0$. Values of L for $R = 0$ and $R \rightarrow 1$ are $L = 1$ and $L \rightarrow 0$, respectively. [The limit $L \rightarrow 0$ as $R \rightarrow 1$ is not obvious from Eq. (1), but can be verified by substitution into the final form of the equation, i.e. into Eq. (2).]

In order to test the validity of the proposed S-N-L relationship further, it was necessary to evaluate the experimental constants a, b, and c. Briefly, the procedure used was to change Eq. (1) to linear form by taking logarithms of the logarithms of both sides of the equation, transposing some terms, using least squares to find the constants, and then reversing the procedure just described to get the relationship back into the same form as Eq. (1). The result is

$$L = 10^{-0.0957\, R^{1.33} (\log N)^{3.17}} \qquad (2)$$

The multiple regression coefficient calculated for the variables of Eq. (2) was found to be 60.6 percent. This means that the equation explains 60.6 percent of the variance in the observed data. Details of the statistical analysis are given in the appendix.

Further investigation of the S-N-P *relationship*—The applicability of the statistical theory of extreme values to the analysis of this concrete fatigue data was also investigated. Freudenthal and Gumbel[10,11] have proposed two distributions of this kind, one called the "linear theory" and the other called the "general theory." They showed that the results of reversed torsional fatigue tests of nickel wire agreed well with the linear theory at high stress levels and with the general theory at lower stress levels. The data did not include any run outs—i.e., specimens which did not fail, but which were not tested beyond a large, arbitrary number of cycles.

In applying these extreme value techniques to analyze the data of Table 1 it was necessary, because of the run outs, to use only part of the data or to attempt to use methods for a truncated distribution.[12] The latter methods enable one to take data covering only part of the distribution and extrapolate it to find the distribution.

Using the first method (i.e. only N-values for failures) the statistics proposed by Freudenthal and Gumbel[10,11] were calculated and the necessary criteria were applied to see if the data fitted either the linear theory* or the general theory* well enough to be represented by them. In each case there was no relationship. Further details of this analysis are given in the appendix.

*The name "linear theory" comes from the fact that the plot, on extremal probability paper, of the probability of survival L, versus the logarithm of the number of cycles to failure, log N, is a straight line. The plot of the "general theory" is a curved line which becomes asymptotic to a lower limit of N, below which failure will not occur.

Fig. 3—*S-N-P diagram for concrete beams in reversed bending*

It was not possible to use the method for truncated distributions described by Hald[12] as this method requires extensive data to predict (extrapolate) the frequency distribution accurately. This would have involved a much more extensive testing progrem than it has been practicable to carry out.

CONCLUSIONS

Regarding the study described, the following conclusions were drawn:

1. In a range up to 20 million cycles, it appears that S-N curves for concrete do not become asymptotic to a particular stress level. The number of cycles to failure N continues to increase as the stress level S is decreased..

2. At 20 million cycles the probability of failure was slightly less than one-half for concrete tested at a stress level of 50 percent of the modulus of rupture.

3. An adequate representation of concrete fatigue data probably should include the probability of failure factor P.

4. The S-N-P relationship for concrete can be obtained graphically by the method used in this study.

5. Eq. (2),

$$L = 10^{-0.0957\,R^{2.33}(\log N)^{2.17}}$$

explains 60.6 percent of the variance among the variables L, R, and N.

6. The fatigue characteristics of the concrete tested cannot be represented by the extreme value functions proposed by Freudenthal and Gumbel.[11]

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APPENDIX

PROCEDURE FOR EVALUATING THE CONSTANTS OF EQ. (1)

The first step in the procedure used to evaluate the constants in the equation

$$L = 10^{-aR^b (\log N)^c} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

was to change it to linear form by taking logarithms of the logarithms of both sides of the equations. It was then written as

$$\log (-\log L) = \log a + b \log R + c \log \log N \quad \ldots\ldots\ldots\ldots\ldots (3)$$

changing variables this gave

$$Y = A + bX + cZ \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4)$$

where $Y = \log(-\log L)$, $A = \log a$, $X = \log R$, and $Z = \log \log N$. As it was desired to predict Z from X and Y (i. e., to predict the cycles to failure for a given stress ratio and a certain probability of survival) the equation was transposed to read

$$Z = A' + B' X + C' Y \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (5)$$

where $A' = -A/c$, $B' = -b/c$, and $C' = 1/c$.

Eq. (4) expresses the regression of Z on X and Y. Because it is more convenient to work with the variables measured from their sample means than with the variables themselves, the relationship was rewritten as

$$z = b' x + c' y \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (6)$$

where $z = Z - \overline{Z}$, $x = X - \overline{X}$, and $y = \overline{Y} - Y$.* From these data the following statistics were calculated:

$$\sum x^2 = 0.137070 \qquad \sum xy = 0.354093$$
$$\sum y^2 = 7.730429 \qquad \sum yz = 2.069986$$
$$\sum z^2 = 0.768163 \qquad \sum xz = -0.032064$$

By means of the least square normal equations

$$b' \sum x^2 + c' \sum xy = \sum xz$$
$$b' \sum xy + c' \sum y^2 = \sum yz$$

the constants b' and c' of Eq. (6) were evaluated. The resulting equation was

$$z = -1.0499\, x + 0.3159\, y \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (7)$$

*Complete tabulation of numerical values of x, y, and z is available on request from ACI headquarters.

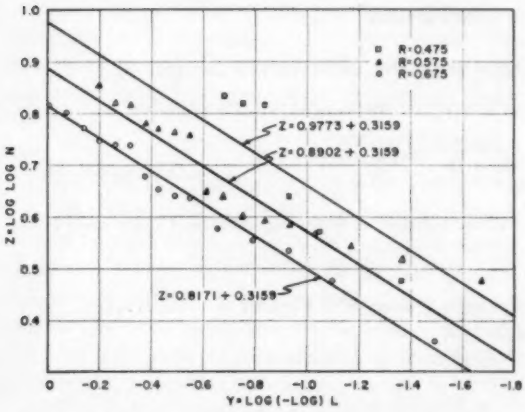

Fig. 4—Comparison of observed and calculated Z-values

or, in the alternate forms

$$Z = 0.6379 - 1.0499\,X + 0.3159\,Y \quad \ldots\ldots (8)$$

and

$$L = 10^{-0.0957\,R^{1.32}(\log N)^{1.17}} \quad \ldots\ldots (2)$$

The latter equations correspond to Eq. (5) and (1) respectively.

To measure the degree of association among the three variables x, y, and z the multiple correlation coefficient was calculated as follows:

Total S.S. $= \sum z^2$	$= 0.768163$
S.S. due to regression $= b' \sum xy + c' \sum yz$	$= 0.282070$
S.S. due to error	$= 0.486093$

$r_{z \cdot xy} = \sqrt{\dfrac{0.282070}{0.768163}} = 0.606$, or $r_{z \cdot xy} = 60.6$ percent, where $r_{z \cdot xy}$ is the multiple correlation coefficient of z on xy.

The goodness of fit of the proposed relationship is illustrated in Fig. 4 which shows both the observed data and the plots of Eq. (8) for the three stress levels used.

INVESTIGATION OF THE EXTREME PROBABILITY FUNCTIONS

As mentioned previously, the probability functions which Freudenthal and Gumbel[10,11] have proposed are referred to as the linear theory and the general theory. "By-eye" fits of straight and curved lines to the observed data, assumed to correspond roughly to the linear and general theories, are shown in Fig. 5 and 6 respectively. The value V_e shown in Fig. 5 is called the characteristic number of cycles and is the N-value corresponding to a probability of survival of 1/e, where e is the base of natural logarithms.

In order to determine if the linear theory may be used to describe the observed data the arithmetic and geometric standard deviations are calculated. Using the arithmetic standard deviation the geometric standard deviation is now estimated by the methods described in References 10 and 11. If the observed and estimated values agree the theory is acceptable. From the statistics given in Table 4 it may be seen that the linear theory did not fit the concrete fatigue data at any of the stress levels.

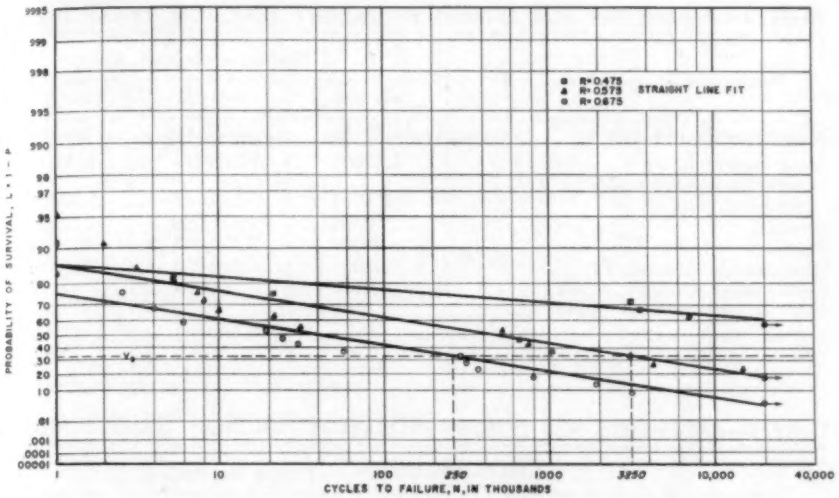

Fig. 5—Plot of fatigue data on extremal probability paper

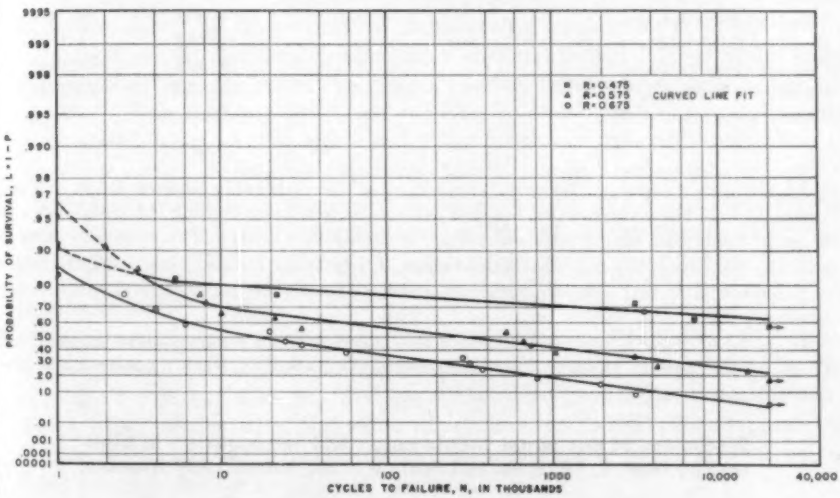

Fig. 6—Plot of fatigue data on extremal probability paper

At the lowest stress level it was possible to find V_s only if the curve was extrapolated. This gave sufficient evidence, however, to reject the theory. For the two higher stress levels the coefficient of variation was so large that it was outside of the range of values covered in References 10 and 11; consequently, no estimated values of the geometric standard deviation are given in Table 4. The linear theory was rejected because of the excessively large coefficient of variation.

TABLE 4—PARAMETERS FOR EXTREME PROBABILITY FUNCTION, LINEAR THEORY

Parameters at given stress level	Testing stress level, R		
	±0.475	±0.575	±0.675
Number of specimens, n	8	16	19
Mean logarithm, $\overline{\log N}$	4.62803	4.85414	4.30948
Observed geometric standard deviation, $S_{(\log N)}$	1.6299	1.70757	1.25110
Mean, $\overline{N}$	1,771,500	1,541,688	372,053
Standard deviation, S_n	2,445,800	3,503,348	803,801
Characteristic number, $\hat{V}_s$	40,000,000+	3,250,000	250,000
Coefficient of variation, S_n/V_s	<0.61	1.08	3.22
Estimated geometric standard deviation, $\hat{S}_{(\log N)}$	<0.37	—	—
Theory acceptable? $S_{(\log N)} \stackrel{?}{=} \hat{S}_{(\log N)}$	No	No	No

TABLE 5—PARAMETERS FOR EXTREME PROBABILITY FUNCTION, GENERAL THEORY

Parameters at given stress level	Testing stress level, R		
	±0.475	±0.575	±0.675
Number of specimens, n	8	16	19
Mean, $\overline{N}$	1,771,500	1,541,688	372,053
Standard deviation, S_n	2,445,800	3,503,348	803,801
Reduced third moment, $\sqrt{b_{1,s}}$	1.0119	3.3242	2.8159
Characteristic number, $\hat{V}_s$	2,188,511	1,080,998	299,068
Sensitivity limit, $\hat{N}_{o,s}$	−1,947,354	−946,039	−269,621
Theory acceptable? $(N_{o,s} \stackrel{?}{>} 0)$	No	No	No

According to the general theory the curve showing the relationship between L and N is not a straight line, but a curve which becomes asymptotic to a lower limiting value of N referred to as the "minimum life," "threshold value," or "sensitivity limit." This sensitivity limit is found by first calculating the reduced third moment, symbolized by $\sqrt{b_{1,s}}$, then using the tables in Reference 11 to find certain parameters, and finally using the parameters to compute the sensitivity limit.

As shown in Table 5 negative sensitivity limits were found for the concrete fatigue data. These values have no physical significance and are sufficient evidence for rejection of the general theory.

Discussion of this paper should reach ACI headquarters in triplicate by Nov. 1, 1958, for publication in the March 1959 JOURNAL.

Title No. 55-14

From a symposium sponsored by ACI Committee 215

Fatigue Behavior of Reinforced Concrete Beams*

By TIEN S. CHANG† and CLYDE E. KESLER‡

SYNOPSIS

This study concerns the fatigue behavior of small reinforced concrete beams, 4 x 6 x 64 in., designed to fail in flexure under static load. For this type of specimen, the results clearly indicate that the magnitude of repeated load determines the mode of failure. Briefly, a low magnitude repeated load generally will result in a flexural failure, fatigue of the steel, while a high magnitude repeated load will result in a shear failure.

INTRODUCTION

Scope

If the static flexure strength of a concrete beam with a low percentage of tensile reinforcement is much lower than the static shear strength of the beam, the fatigue strength of the beam will most probably be governed by the fatigue life of the tensile reinforcement. The governing mechanism of such a failure will be different from the fatigue behavior of beams failing in shear as discussed in a previous paper.[1] When the static shear and flexure strength of a concrete beam are approximately equal, the mechanism of failure becomes quite complicated. A beam of this type can fail by fatigue of the reinforcement or in shear by either a diagonal cracking failure or shear-compression failure.

The main objective of this investigation is to study the complicated fatigue behavior of a reinforced concrete beam which will fail in flexure at a static load only slightly less than that which would produce a shear failure.

Outline of tests

A total of 25 beams having a 4 x 6 in. cross section and 1.02 percent of tensile reinforcement were tested under repeated loading on a span of 60 in. The effective depth was 5.38 in. The specimens were loaded at the third points. Three other beams with the same dimensions and steel percentage as the fatigue specimens were loaded to failure under static loads applied at the third points and on a simple span of 60 in.

Remaining pieces of beams broken as a result of the fatigue loading, as well as the beams unbroken after approximately ten million cycles of repeated

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loading, were loaded to failure statically to determine the damage due to repeated loading.

MATERIALS

All beams were made with Type I portland cement and Wabash River natural aggregates. The fine aggregate had a fineness modulus of approximately 3.0 and the coarse aggregate was well graded and had a maximum size of about 1 in. Both aggregates passed the usual specification tests.

The reinforcement consisted of #3 deformed bars of intermediate grade billet steel meeting requirements of ASTM A 305. The average yield point was about 45,000 psi, the ultimate strength about 70,300 psi, and the modulus of elasticity about 27,000,000 psi.

EQUIPMENT AND TESTING PROCEDURE

Fatigue tests of beams were made in a specially designed machine which applied a sinusoidal loading at the third points of a 60-in. simple span at the rate of 440 cycles per minute. The testing machine[2] was of the type which applies a constant deformation to the specimen. Thus to maintain a constant load it was necessary to make frequent checks and adjust the machine as needed. An automatic shut-off was used to stop the machine when the specimens failed. The number of cycles to failure was recorded on a revolution counter.

Static tests of beams were made in a universal testing machine in which the beams were loaded to failure in from 10 to 60 min.

RESULTS

Notation

a = shear span of a simply supported beam measured from center of support to center of loading plate, in.

a' = shear span of a simply supported beam measured from center of support to edge of loading plate, in.

A_s = area of tensile reinforcement, sq in.

b = width, in.

d = effective depth, in.

f_c' = concrete strength of 6 x 12-in. cylinders, psi

f_y = yield stress of steel, psi

f_u = ultimate strength of steel, psi

M = critical moment developed in a simply supported beam, in.-lb

M_y = flexure moment a concrete beam can carry when the reinforcement is stressed at its yield point, in.-lb

M_f = flexure moment a concrete beam can carry when the reinforcement is stressed to its ultimate strength, in.-lb

M_s = maximum shear moment, in.-lb

p = steel percentage

V = maximum shear, lb

V_c = static diagonal cracking strength of a simply supported beam with tensile reinforcement, lb

Fatigue tests

Fourteen of the 25 beams tested under repeated loading failed by fatigue of the tensile reinforcement in the maximum moment region. In all cases these beams developed tensile cracks and in many cases both tensile and diagonal cracks developed before final failure occurred, as shown in Fig. 1a and 1b.

Four of the beams tested failed by diagonal cracking, i.e., by the simultaneous development of a diagonal crack and the destruction of the compression zone as shown in Fig. 1c.

Four of the beams tested failed by shear-compression, i. e., by first developing a major diagonal crack followed by destruction of the compression zone after many additional cycles of repeated loading. This type of failure is shown in Fig. 1d.

The remaining three beams were taken out of the fatigue machine before failure. Some tensile and diagonal cracks were observed.

Data concerning the properties of the beams, magnitudes of loading, number of cycles to diagonal tension cracking, and/or failure are listed in Table 1.

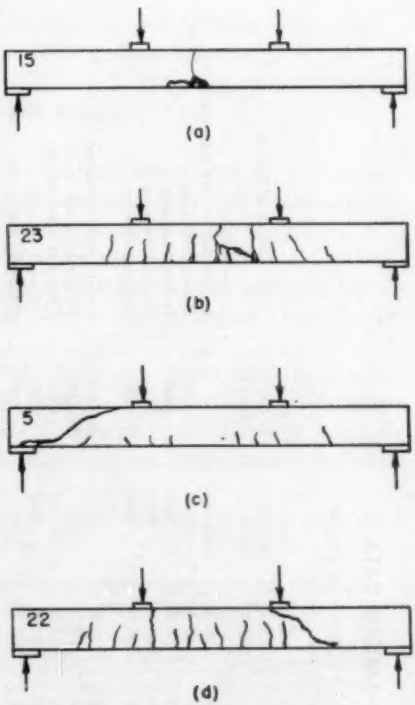

Fig. 1—Types of failures

Static tests

Three beams of 4 x 6-in. cross section were tested to failure statically under third-point loading on a simple span of 60 in. The beams failed when the reinforcement was well into the strain hardening region. No diagonal cracks were observed. The moment at which the reinforcement yielded and the maximum moment correspond very closely to that given by the following formulas:

$$M_y = A_s df_y \left[1 - \tfrac{1}{2}\left(\frac{A_s f_y}{bdf_c'}\right)\right] \quad \ldots\ldots(1)$$

and

$$M_f = A_s df_u \left[1 - \tfrac{1}{2}\left(\frac{A_s f_u}{bdf_c'}\right)\right] \quad \ldots\ldots(2)$$

Supplementary static tests

The best remaining piece of each of the beams broken by repeated loading was tested to failure statically under center-point loading while simply supported. The results are given in Table 2. The span in each case may be determined from the table by noting the value of a, the distance from the center of the loading plate to the center of the support.

TABLE 1—FATIGUE DATA

$b = 4$ in., $d = 5.38$ in., $p = 1.02$ percent

Specimen No.	f'_c, psi	M_f* in.-lb	$\frac{M}{M_f}$, percent	M_c, in.-lb	$\frac{M}{M_c}$, percent	V_c lb	$\frac{V}{V_c}$ percent	Cycles to diagonal cracking	Cycles to final failure	Mode of failure
1	5330	77,900	45.0	64,000	52.0	4280	40.8	4,965,800	—	Did not fail at 6,966,700 cycles
2	6200	78,600	72.5	67,800	79.6	4480	63.5	10,000	23,500	Shear-compression
3	5980	78,500	68.8	67,000	76.6	4440	60.8	3000	3000	Diagonal cracking
4	4950	77,500	72.9	62,000	86.5	4190	67.5	6500	6500	Diagonal cracking
5	5250	77,800	72.6	63,700	84.3	4280	66.0	1800	1800	Diagonal cracking
6	4930	77,400	51.7	61,800	61.4	4190	47.8	—	483,200	Fatigue of reinforcement
7	5340	77,900	57.7	65,200	65.5	4290	52.5	—	395,600	Fatigue of reinforcement
8	5500	78,100	72.4	64,800	82.8	4330	65.2	missed	8000	Fatigue of reinforcement
9	5350	77,900	72.6	64,200	83.5	4290	65.8	2600	2600	Diagonal cracking
10	4430	76,900	58.5	59,200	72.2	4040	55.7	—	47,100	Shear-compression
11	5660	78,200	38.4	65,600	43.4	4370	34.3	—	—	Did not fail at 20,472,900 cycles
12	5550	78,100	73.0	65,400	82.8	4340	65.6	10,400	23,200	Shear-compression
13	4750	77,300	58.2	60,800	70.2	4140	54.4	—	172,100	Fatigue of reinforcement
14	5500	78,100	64.2	64,800	73.3	4330	57.8	—	445,500	Fatigue of reinforcement
15	6080	78,500	63.7	67,300	70.5	4460	56.2	—	502,800	Fatigue of reinforcement
16	5620	78,200	51.2	65,400	58.1	4360	46.0	—	—	Did not fail at 576,100 cycles
17	4530	77,100	52.0	59,600	63.7	4060	49.3	<10,000	713,500	Fatigue of reinforcement
18	5200	77,800	64.3	63,300	75.0	4260	58.7	—	420,200	Fatigue of reinforcement
19	5980	78,500	57.3	65,400	65.4	4440	50.7	—	463,500	Fatigue of reinforcement
20	6300	78,700	72.5	68,200	79.4	4500	63.3	missed	3200	Fatigue of reinforcement
21	6800	79,000	72.2	69,200	78.2	4590	62.2	<10,000	25,800	Fatigue of reinforcement
22	6400	78,800	71.0	68,400	78.0	4530	61.8	<10,000	30,000	Shear-compression
23	6100	78,100	73.0	67,500	80.0	4460	64.0	<10,000	70,000	Fatigue of reinforcement
24	6300	78,700	72.5	68,200	79.2	4500	63.3	<10,000	150,900	Fatigue of reinforcement
25	6300	78,700	64.8	68,200	71.1	4500	56.7	<10,000	140,500	Fatigue of reinforcement

*Assuming the stress in the reinforcement is at the maximum when tested to failure statically.

The three beams which did not fail under fatigue loading were also tested to failure under center-point static loading while simply supported.

ANALYSIS OF TEST RESULTS

Modes of failure

Under static loads and for the span-depth ratio used, $a/d = 3.72$ where $a = 20$ in., in the repeated load tests, the beams had a very low yield moment. However, due to the low percentage of tensile reinforcement, the concrete in the compression zone was still capable of carrying higher stresses. In fact, the beam, under static load, could carry a maximum flexure moment corresponding to the ultimate strength of the reinforcement before the concrete would crush.

The static diagonal cracking loads of the beams, as predicted by Eq. (3), were found to be approximately 10 to 15 percent higher than the maximum loads corresponding to the static ultimate flexure moment carrying capacity of the beams. Under static loads, therefore, the beams would exhibit a flexure failure.

However, under repeated loading, the matter became quite complicated. The beams could fail in the following ways:

1. *Fatigue of reinforcement, no diagonal tension cracking*—Seven beams developed tensile cracks in the early stages of loading and then failed due to fatigue of reinforcement in the maximum flexural moment region.

To fail in this manner, the beam had to have a relatively high static diagonal cracking load as compared to the static maximum flexural carrying capacity. The beam had to be loaded to a high enough maximum repeated loading to cause fatigue failure in the reinforcement. However, the maximum repeated load could not be so high as to be able to cause diagonal cracking before the reinforcement broke. Beams 6, 7, 13, 14, 15, 18, and 19 failed in such a fashion.

2. *Fatigue of reinforcement, with diagonal cracking*—Seven beams developed tensile and diagonal cracks and then failed due to fatigue of reinforcement in the region of maximum flexural moment.

To develop this mode of failure, the beam had to have a maximum diagonal cracking load comparable to the maximum flexural load. The maximum repeated load had to be high enough to be able to cause diagonal cracking before the reinforcement broke. The maximum repeated load had to be lower than the load corresponding to the static shear moment of the beam, and also to a certain percentage of the static load so that the beam would not fail in shear-compression before the reinforcement failed. Beams 8, 17, 20, 21, 23, 24, and 25 failed in this way.

3. *Diagonal cracking*—Four beams failed simultaneously by diagonal cracking and shear-compression after first developing some tensile cracks.

To develop this mode of failure, the beam had to have a quite high repeated load so that the fatigue lives in diagonal cracking and shear-compression were identical. In such cases there was not time for a fatigue failure to develop in the reinforcement. Beams 3, 4, 5, and 9 failed in this manner.

4. *Shear-compression*—Four beams developed tensile and diagonal cracks, and then failed by the destruction of the concentrated compression zone later as a result of the diagonal cracking.

To develop this mode of failure, the beam had to be loaded to a maximum repeated load which would cause a diagonal crack before fatigue of the reinforcement and yet

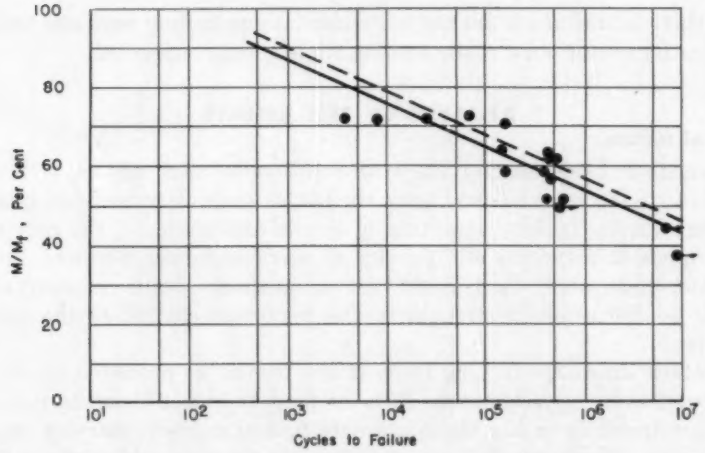

Fig. 2—Modified *S-N* diagram for beams failing in flexure due to repeated loading

not be so high as to cause failure at the time the diagonal crack developed. The maximum repeated load also had to be able to cause shear-compression failure before the reinforcement failed in fatigue. Beams 2, 10, 13, and 22 failed in this manner.

Modified *S-N* diagram for beams which failed due to fatigue of reinforcement

In Fig. 2 the maximum percentage of the repeated loading, based on the static maximum flexural moment, was plotted semilogarithmically against the number of cycles to cause the final fatigue failure of the reinforcement for each

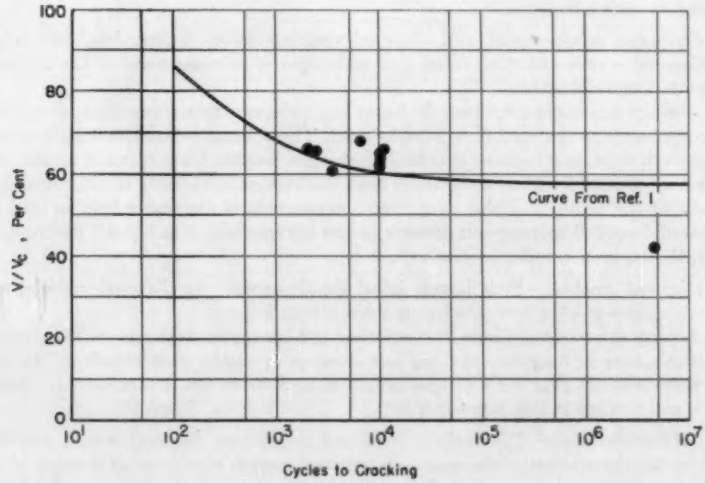

Fig. 3—Comparison of diagonal cracking life with results of previous investigation (Reference 1)

beam which failed due to fatigue of the reinforcement. A 50 percent probability line was drawn through the plotted points. The fatigue factor was found to be approximately 0.43. The fatigue factor as used in this paper is the ratio to static strength of the maximum repeated load the specimen could sustain to ten million cycles without failure. The load may be expressed as the total load, shear, or moment on the specimen or portion of it under consideration. The fact that the damage tests (Table 2) and actual static tests showed the actual maximum flexural moments to be generally 5 percent lower than the flexural moments based on the assumption that the stresses in the reinforcement were at their maximum values, suggested that perhaps the stresses in the reinforcement were slightly less than the maximum at failure when tested statically. In such a case, the 50 percent probability line probably should be raised by 3 to 5 percent as indicated by the dotted line shown in Fig. 2. The fatigue factor thus found was approximately 0.48.

Modified *S-N* diagram for diagonal cracking

The percentage of the maximum static diagonal cracking load at which the specimens were loaded in fatigue was plotted semilogarithmically against the diagonal cracking life of each beam which developed such a crack. The static diagonal cracking load was computed from the following expression:

$$\frac{V_c}{bd} = 1.90 f_c'^{0.33} (1 + 0.00562p^{1.8}) \left(1 + 0.685 \frac{d^{1.42}}{a^{1.42}}\right)\left(1 + \frac{13.4}{d^{2.6}}\right) \quad \ldots\ldots\ldots\ldots (3)$$

This expression and the one given below for failure in shear-compression were developed by dimensional analysis and are explained in Reference 1 together with convenient nomographs for their solution.

The 50 percent probability curve for cracking previously obtained by the authors from tests on beams failing in shear[1] was also drawn on the same graph. A good fit was found for the two sets of data as shown in Fig. 3. This means that the modified *S-N* diagram for cracking developed previously is also valid for the beams tested in this investigation.

The grouping of the data in Fig. 3 is not accidental. For the beams tested, diagonal cracking can only occur if the load is high enough, $V/V_c = 60$ percent, to cause the crack to form before the steel failed. When the repeated load was lower, the diagonal crack would not form and although the beam might fail in flexure it would not be plotted on Fig. 3.

Modified *S-N* diagram for beams failing in shear

The percentage of the maximum static shear moment at which the specimens were loaded in fatigue was plotted against the logarithm of the life to failure in Fig. 4. The maximum static shear moment was computed as follows:

$$\frac{M_c}{bd^2} = 21.3 f_c'^{0.35} (1 + 0.188p^{1.62}) \left(1 + \frac{17.8}{d^{3.25}}\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots (4)$$

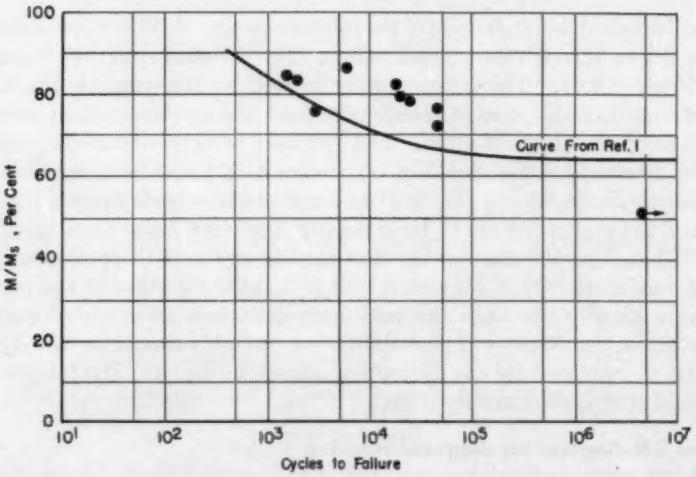

Fig. 4—Comparison of fatigue life of beams failing in shear with results from previous investigation (Reference 1)

The fatigue data for beams failing in shear in this investigation was also compared with the modified *S-N* diagram obtained for the beams from the previous investigation.[1] Again a good fit was observed for both sets of data, as shown in Fig. 4. The data are again grouped for the same reason as above since a shear failure cannot occur unless a diagonal crack has formed.

TABLE 2—STATIC DAMAGE TESTS

$b = 4$ in., $d = 5.38$ in., $p = 1.02$ percent

Specimen No.	f_c', psi	a^*, in.	$M_{f(test)}$ lb-in.	$\frac{M_{f(test)}}{M_{f(calc)}}$	$M_{s(test)}$ in.-lb	$\frac{M_{s(test)}}{M_{s(calc)}}$	$V_{c(test)}$ lb	$\frac{V_{c(test)}}{V_{c(calc)}}$	Mode of failure
1	5330	30.5	74,000	0.95	—	—	—	—	Flexure
2	6200	19.5	81,500	1.04	—	—	—	—	Flexure
3	5980	19.5	73,500	0.94	—	—	—	—	Flexure
4†	4950	20.5	—	—	65,800	1.06	—	—	Shear-compression
5	5250	20.0	73,000	0.94	—	—	—	—	Flexure
6	4930	16.5	—	—	—	—	8200	0.94	Diagonal cracking
7	5340	13.5	—	—	60,500	0.93	—	—	Shear-compression
8	5500	30.5	68,700	0.88	—	—	—	—	Flexure
9	5350	20.5	72,800	0.94	—	—	—	—	Flexure
10†	4430	21.5	—	—	62,200	1.05	—	—	Shear-compression
11	5660	30.5	76,500	0.98	—	—	—	—	Flexure
12†	5550	19.5	—	—	70,000	1.07	—	—	Shear-compression
13	4750	12.5	—	—	64,500	1.06	9500	1.05	Shear-compression
14	5500	11.5	—	—	57,700	0.89	10,000	1.01	Shear-compression
15	6080	13.0	—	—	67,000	1.00	10,000	1.01	Shear-compression
16	5620	10.5	—	—	69,300	1.06	8800	0.88	Shear-compression
17	4530	13.5	—	—	56,800	0.96	8750	0.97	Shear-compression
18†	5200	18.0	—	—	71,500	1.13	—	—	Shear-compression
19	5980	12.5	—	—	67,500	1.02	8300	0.86	Shear-compression
20†	6300	30.5	—	—	67,500	0.99	—	—	Shear-compression
21†	6800	12.5	—	—	70,700	1.02	—	—	Shear-compression
23	6100	15.5	—	—	—	—	9050	0.95	Diagonal cracking
24	6300	12.5	—	—	62,100	0.91	9000	0.85	Shear-compression
25	6300	12.5	—	—	63,300	0.93	9300	0.88	Shear-compression

*a is measured from the center of the loading plate to the center of the nearest support. The width of the loading plate is 2-in.

†These beams had developed diagonal cracks under fatigue loading.

Damage tests (Table 2)

The results of the static tests on the remaining pieces of beams that failed in fatigue or on beams that did not fail in fatigue showed that no serious damage was done to any beam by fatigue except near the locations where cracks and failures occurred.

For small span-depth ratios the beams failed in diagonal cracking or shear-compression. For larger span-depth ratios, a'/d larger than 3.5, where a' is the distance from the center of the support to the edge of the loading plate, the beams failed in flexure provided they had not developed diagonal cracks under repeated loading. For beams with previously developed diagonal cracks, the mode of failure was always in shear-compression.

SUMMARY AND CONCLUSIONS

The fatigue behavior of concrete beams with 1.02 percent steel, was investigated. Due to the choice of steel percentage, span-depth ratio, concrete strength, and magnitude of repeated load, four modes of failure were obtained: (1) fatigue of reinforcement without diagonal cracking; (2) fatigue of reinforcement with diagonal cracking; (3) diagonal cracking; and (4) shear compression.

From the results and analysis of this investigation, the following conclusions may be drawn with regard to the particular beams tested.

1. For beams of high static diagonal cracking load as compared to the static maximum flexural carrying capacity, the beams will generally fail due to fatigue of the reinforcement, provided the percentage of maximum repeated load will not cause diagonal cracking during loading, i.e., V/V_c will be approximately equal to or less than 0.60.

2. The beams may still fail in flexure (fatigue of the reinforcement) after developing diagonal cracks, provided the percentage of maximum repeated load does not cause shear-compression failure before the reinforcement breaks. In this case M/M_s must be approximately equal to or less than 0.70.

3. The beams will fail in shear-compression, if the loading first caused a major diagonal crack and if the fatigue life for shear-compression under this particular percentage of repeated load is smaller than the fatigue life of the reinforcement, i.e., M/M_s will probably be equal to or greater than 0.70.

4. The beams will fail in diagonal cracking, if the percentage of repeated loading will cause simultaneous diagonal cracking and shear-compression failure before the reinforcement breaks. In this case V/V_c will be equal to or greater than 0.60 and M/M_s must be equal to or greater than 0.80.

5. The cracking lives of the beams tested for this investigation can be predicted from the modified S-N diagram for cracking developed previously by the authors for beams weak in shear.

6. The failure lives of beams in shear for this investigation can be predicted from the modified *S-N* diagram for shear failure developed previously by the authors for beams weak in shear.

7. Due to fatigue loading, the beams will normally be seriously damaged only at the points where cracks and failures occur and not in the remaining pieces of the beams.

The results of this study clearly indicated that much additional information is needed to determine their general applicability. Dividing points given for the various modes of failures for the particular beams studied may vary when any of the parameters are changed beyond those changes considered. However, the development of a general criterion of failure appears feasible. Additional data can be most economically obtained by a combination of analytical and experimental methods.

ACKNOWLEDGMENT

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Title No. 55-15

From a symposium sponsored by ACI Committee 215

Fatigue Properties of Concrete Beams*

By THOMAS E. STELSON† and JOHN N. CERNICA‡

SYNOPSIS

Eleven concrete beams with regular tensile reinforcement were tested under repeated load applied 320 times per min. The beams were identical except for a slight increase in concrete strength during the period of testing. The ACI elastic-design load for the test beams was 2610 lb. The ultimate load was 8800 lb. The load causing fatigue failure in 500,000 cycles was 5720 lb. Thus, the ultimate load was 3.37 times the design load and the fatigue load was 2.18 times the design load.

INTRODUCTION

A few investigations have been made of the behavior of reinforced concrete beams subjected to repeated load. For a summary of the previous study of fatigue properties of reinforced concrete beams, see "Fatigue of Concrete—A Review of Research," by Gene M. Nordby, appearing on p. 191 of this JOURNAL issue.

The purpose of the present study was to develop a better understanding of the behavior of reinforced concrete beams subjected to repeated loads and to show how these results compare with previous tests of cylinders under repeated loads.

More information on the fatigue properties of concrete is available from cylinder tests than from beam tests. A maximum design stress of 45 percent of the short-time ultimate strength may be appropriate for safety from fatigue failure in concrete members under axial load and uniform axial stress. The minimum safety factor from repeated overstress would be about $55/45 = 1.22$, for a minimum stress of zero. With a minimum stress greater than zero, the safety factor would be increased.

Applying information from cylinder tests to the design of beams, however, may be misleading. When a stress variation exists over the cross section as in a beam, the fatigue properties of the member may be far different from the fatigue properties of the extreme fiber. Relationships between cylinder strength and beam strength under short-time static load are complex. Under repeated load such relationships are even more difficult to evaluate.

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TESTING PROGRAM

Specimens

The beam specimens used for this study were of rectangular cross section, 5.0 in. wide, 5.5 in. deep and 7 ft long, reinforced with two #5 high bond, hard grade rods whose centroids were 4.25 in. from the compression face. The bars were placed 2.5 in. apart, center to center.

One batch of commercially supplied truck mixed concrete was used. It had a cement:fine aggregate:coarse aggregate ratio by weight of 1:2:3.5 with a maximum aggregate size of 0.5 in. and a slump of 3.5 in. The 28-day cylinder strength was 3800 psi when the test program began. At the end of the test program the cylinder strength had increased to 3900 psi.

The beams were intentionally overreinforced to insure failure of the concrete rather than the steel. The yield point of the steel was 70,000 psi (0.2 percent permanent deformation) and its ultimate strength was 128,000 psi.

The ACI elastic-design load* for these specimens is 2610 lb (total on two third points of 72-in. total span). The design load is controlled by the maximum compressive stress $0.45 \times 3800 = 1710$ psi. At the design load the ACI shearing stress* is 73 psi and the allowable shearing stress is 90 psi. The computed steel stress is 14,400 psi.

To determine the ultimate strength of the beams, several beams were tested under static load applied at the third points of a 72-in. span. The ultimate load was measured to be 8800 lb total on the two third points.

Apparatus

Photographs of the apparatus are shown in Fig. 1 and 2. A frame of I-beams welded together supports the test beams, a loading shaft, a belt-driven pulley system, and an electric motor.

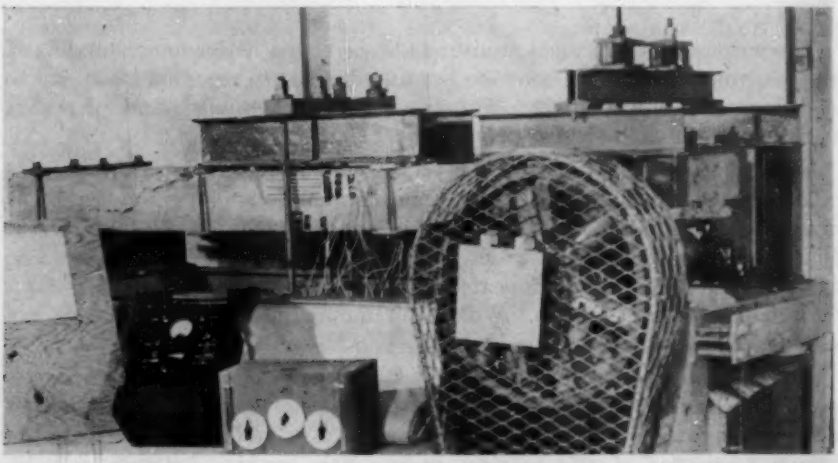

Fig. 1—Side view of the apparatus showing typical beam failure in testing position

The frame is of rectangular shape 5 ft high, 8 ft long, and 3 ft wide. The loading shaft has two 5 in. long sections, 1 in. eccentric and out of phase with each other by 180 deg as shown in Fig. 2.

Fig. 3 is a schematic diagram showing the loading and operating arrangement. As the shaft turns, the eccentric portion of the shaft lifts one end of the beam applying the load. The other end rests on a fixed support. Two calibrated yokes (one on each side of the concrete beam) pull down on an I-beam which transmits load to the concrete beam at the third points. The

*Based on "Building Code Requirements for Reinforced Concrete (ACI 318-56)."

yokes have reduced sections where SR-4 electric resistance strain gages (on opposite sides) are installed. The bars were calibrated, obtaining a relationship between load and the strain in the gages.

A plate supports the load end of the concrete beam. It serves as a guide for the vertical movement of the beam and also is adjustable for applying the desired minimum load.

The eccentric portions of the load shaft consist of oil bearings covered by a ½ in. thick steel bushing. In applying load the horizontal thrust between the bushing and the concrete beam is very small.

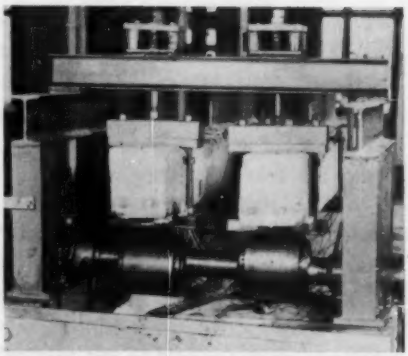

Fig. 2—End view of the apparatus showing load shaft and minimum load control

Testing procedure

The beams were simply supported over a 6-ft span. One end rested on a fixed support and the other was movable and adjustable.

1. The eccentric portion of the load shaft was turned so that the point of maximum throw would contact the beam. Then the yokes were tightened to the desired maximum load.

2. The shaft was then turned 180 deg and the plate supporting the movable end tightened and adjusted until the minimum load (as read in the yokes) was obtained. In this manner any desired range of load could be obtained.

3. A retightening of the yokes was made as plastic deflection occurred in the beam from repeated load. The testing machine was of the constant deflection type, but by periodic adjustment of the maximum and minimum load, the operation was essentially under constant load until rapid degeneration of the beam at failure.

To insure uniform distribution of load, plaster of paris was used at the points of load and support.

MODE OF FAILURE

All beams under both static and repeated load failed by cracking diagonally along a line extending through the outer third of the beam from the load point to reaction point. The failure crack would extend from the load point down to the reinforcing bars and then follow the bars out to the end of the beam. A typical failure crack is shown at the left end of the beam in Fig. 1.

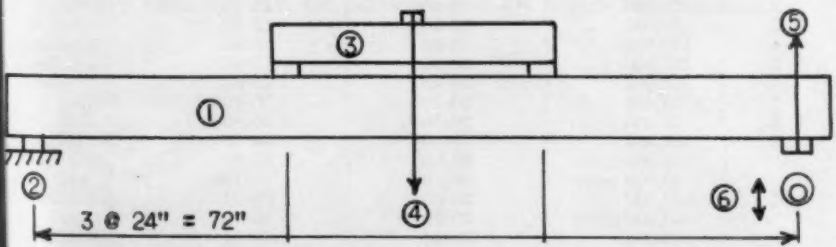

Fig. 3—Schematic diagram of apparatus: (1) concrete beam specimen; (2) fixed support; (3) load beam; (4) adjustable yoke to establish maximum load during cycle; (5) adjustable yoke to establish minimum load when eccentric is not in contact; (6) eccentric lifts end of beam changing load from minimum to maximum and back to minimum

The failures were usually sudden, sometimes almost explosive, under both static and repeated load. In one instance, the upper corner of the beam above the failure crack landed 3 ft from the base of the testing machine.

The beams were thoroughly examined for cracks about every half hour or 10,000 cycles when the machine was stopped, measurements were taken, and the load corrected. Frequent casual examinations for cracks were made often during the half-hour intervals, particulalry during the early cycles. If any cracks were found, the machine was stopped, measurements were taken, the loads were corrected, and the test was resumed.

With one exception, the beams could withstand only a few, if any, cycles after cracking. Upon cracking, the load would often be greatly relaxed. Attempts to increase load to the test value would then cause static failure of the beam.

SUMMARY AND CONCLUSIONS

The test results are shown in Table 1. Beams 9, 10, and 11 were subjected to increasingly larger maximum loads. For these beams the cycles of load refer to additional cycles after the load range was increased.

The ACI elastic-design load for these beams is 2610 lb (total on the two third points). The ultimate short-time static load is 8800 lb or 3.37 times the design load.

When the peak in the load cycle was equal to or more than 65 percent of the ultimate load, the beams always failed in fatigue. When the peak in the load cycle was equal to or less than 60 percent of the ultimate load, the beams did not fail in fatigue when subjected to more than 500,000 cycles. Thus, the

TABLE 1—TEST RESULTS

Beam	Cycles of load	Range of load, percent of ultimate*	Range of nominal elastic stress, percent of ultimate†	Remarks
1	66,000	10–65	15–98	Failed
2	323,000	10–65	15–98	Failed
3	570,000	10–55	15–83	No failure
4	570,000	10–55	15–83	No failure
5	757,000	10–45	15–68	No failure
6	757,000	10–45	15–68	No failure
7	32,000	20–70	30–108	Failed
8	20,800	20–70	30–108	Failed
9	503,000	20–50	30–76	No failure
	520,000 more	20–60	30–91	No failure
	53,000 more	20–70	30–108	Failed
10	503,000	20–60	30–91	No failure
	33,000 more	20–70	30–108	Failed
11	542,000	20–40	30–61	No failure
	676,000 more	20–50	30–76	No failure
	286,000 more	20–65	30–98	Failed

*Ultimate static load measured to be 8800 lb total on two third points of identical beams.
†Ultimate static stress measured to be 3800-3900 psi for 6 x 12-in. test cylinders cast from the same batch of concrete and cured with the beams.

fatigue strength of the test beams at 500,000 cycles is more than 60 percent but less than 65 percent of their ultimate strength. This is $0.60 \times 3.37 = 2.02$ to $0.65 \times 3.37 = 2.18$ times the design load for the beams.

The beams could withstand thousands of cycles of load with a nominal extreme fiber stress by elastic analysis in excess of the ultimate strength of the concrete as measured by cylinder tests. Note the test data for Beams 7, 8, 9, and 10 when the nominal elastic stress in the extreme fiber was 108 percent of the concrete strength. The fatigue strength of the test beams is more than twice their ACI elastic-design load.

Van Ornum and Mehmel have reported loads for the fatigue failure of prisms of 55 and 47 to 60 percent of the ultimate strength, or 1.22 and 1.04 to 1.33 times the present design stress of 45 percent of the ultimate strength. Thus, the beams that were tested show much greater safety against fatigue failure than do compression members by present elastic design criteria. Using the results of cylinder tests for application to the design of beams subjected to repeated load can be misleading. The stress gradient and variation in beams must reduce the likelihood of their fatigue failure.

The beams failed in shear (diagonal tension) under both static and repeated loads even though the shearing stress at design load is only 82 percent of the allowable shearing stress. Thus, shearing stresses may be more critical than compressive stresses in beams subjected to repeated loads and a beam whose design is controlled by shear would show less relative fatigue strength.

ACKNOWLEDGMENTS

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Dean F. T. Mavis, College of Engineering, University of Maryland, provided advice and encouragement and administered the investigation in his former position as Professor and Head, Department of Civil Engineering, Carnegie Institute of Technology.

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Title No. 55-16

The series on air voids continues:

Origin, Evolution, and Effects of the Air Void System in Concrete. Part 2—Influence of Type and Amount of Air-Entraining Agent*

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SYNOPSIS

Air-entraining agents of differing chemical composition produce air voids of different size, distribution, and spacing. The amount of agent also has an effect on these parameters, in general, reducing the air void size and spacing when used in increasing amounts. Air, entrained by an effective agent, in the amount recommended by ACI Committee 613, or greater, will usually provide a satisfactory air void system and satisfactory resistance of concrete to freezing and thawing. In general, the factors which tend to reduce spacing also tend to increase freezing-thawing resistance of concrete.

INTRODUCTION

The normal work of the Bureau of Reclamation requires the testing and evaluation of many air-entraining agents for use as admixtures in concrete of various projects. All air-entraining agents proposed for such use are tested for compliance with the major requirements of ASTM C 260 (specifications for air-entraining admixtures for concrete), and during these tests relatively great differences in the behavior of various air-entraining agents have been noted. For example, one agent in powder form was found to develop a system of voids significantly different from that developed by the same agent used in solution form. Another agent, manufactured outside the United States, entrained air readily but lost a great percentage during the compaction process. These differences in behavior stimulated interest which resulted in the investigations reported here.

Part 1†† of this series of reports discussed various factors that influence the development of an air void system in concrete during mixing and cause progressive alteration of the size and spacing of the air bubbles in the interval

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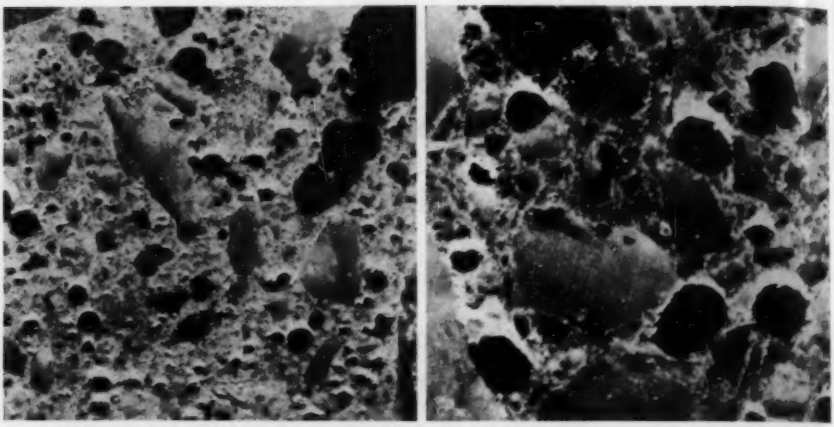

Fig. 1—Air void systems of concretes containing approximately 3.5 percent air. Agent J used in concrete at left, and Agent L at right. Figures are photomicrographs of polished concrete surfaces (magnification about 15.3 X)

before hardening. The void systems developed in response to use of air-entraining agents of various compositions and in varying concentration in the concrete mixture are evaluated in this part of the series, and the effect of the resulting variations on the resistance of the concrete to freezing and thawing is discussed below. The methods and principles by which the parameters of the air void system in hardened concrete were measured are essentially those reported by Brown and Pierson.[12] A summary of the procedures used in obtaining air void parameters for this series of papers will appear in Part 4.

TYPE OF AIR-ENTRAINING AGENT

In all of these investigations the air-entraining agents were employed as admixtures. Numerous organic materials may be used for this purpose. Little is known concerning the fundamental relationships, but it appears (Fig. 1) that different substances will be more or less effective air-entraining agents in mortar or concrete because of differing elasticity of the film produced, differing reduction of surface tension, differing adhesion of the bubbles to particles of cement or aggregate, and differing permeability of the film to the constituents of air* at the concentration used to produce the desired air content.

An air-entraining agent performs more effectively, the better it acts as follows in the solution produced by early hydration of portland cement: (1) develops a film of high elasticity at the air-water interface; (2) reduces surface tension; (3) inhibits transmission of air across the air-water interface; (4) resists thinning or deterioration with time; (5) causes adhesion of the minute air bubbles to particles of portland cement; and (6) does not significantly impair the properties of mortar or concrete. Adequate performance in

*The permeability of the film may vary significantly with respect to oxygen, nitrogen, and carbon dioxide.

TABLE 1—EFFECT OF AIR-ENTRAINING ADMIXTURES ON THE VOID SYSTEM IN CONCRETE*

Air-entraining admixture	Quantity of air-entraining admixture per cu yd of concrete	Water-cement ratio, by weight	Air content, percent, by pressure method	Void parameters determined microscopically					Freeze-thaw expansion per cycle, millionths
				Air content, percent	Chord intercept, in.	Specific surface, in.$^{-1}$	Voids intercepted per in.	Spacing factor, $\bar{L}$, in.	
A	110 ml	0.45	5.6	5.9	0.0042	952	14.22	0.0046	0.0
B (solution)	1032 ml	0.45	4.9	3.5	0.0048	833	7.40	0.0066	0.0
B (powder)	111 g	0.45	4.8	4.8	0.0059	678	8.05	0.0071	0.0
C	147 ml	0.45	5.1	4.6	0.0045	889	10.14	0.0058	0.4
D	109 ml	0.45	5.4	5.1	0.0035	1143	14.57	0.0040	0.0
E	1176 ml	0.45	4.9	5.0	0.0061	656	8.19	0.0073	0.0
F	1838 g	0.45	4.6	5.2	0.0061	656	8.48	0.0071	0.1
G	738 g	0.45	5.1	4.1	0.0065	615	6.28	0.0082	0.0
H	3665 ml	0.45	4.7	3.4	0.0050	800	6.87	0.0071	0.2
I	9398 ml	0.45	2.7	2.8	0.0089	449	3.08	0.0139	—
J	770 ml	0.45	5.1	4.0	0.0038	1050	10.43	0.0051	0.0
K	2192 ml	0.45	5.0	5.8	0.0058	690	9.85	0.0065	0.1
L	411 ml	0.45	4.8	2.3	0.0084	476	2.74	0.0140	6.0
B (solution)	52 ml	0.65	4.9	5.7	0.0063	635	9.05	0.0062	0.0
J	204 ml	0.65	4.7	5.2	0.0051	784	10.19	0.0055	0.0
L	300 ml	0.65	4.8	5.9	0.0112	357	5.26	0.0101	60.0

*Paste content 24.4 to 26.3 percent for $W/C = 0.45$ and 21.2 to 22.5 percent for $W/C = 0.65$.

the first, fourth, and fifth of these categories is essential for satisfactory results. Reduction in surface tension may however vary within wide limits. Glantz and co-workers* found that, among four agents tested, the concentration of agent required to produce an air content of 22 percent in mortar caused a reduction of from 1 to 18 dynes per cm in the surface tension of water. At higher concentration (100 g per l) the surface tension of all of the water-agent mixtures ranged from 31 to 36 dynes per cm.

Little is known about the relative permeability to gases of films of polar molecules adsorbed at air-water interfaces. To the extent that the film reduces the transfer of air below the rate characteristic of pure water (or the water phase in concrete), the air-entraining agent will retard the rate at which α (specific surface of voids) would otherwise decrease with time. According to Clark and Blackman[11] this property of adsorbed films is fundamental to the stability of foams.

Test program

Microscopical examination of concrete specimens demonstrates that the air void system indeed does vary with the nature of the air-entraining agent, even if the other properties of the mixture are held constant. To evaluate the influence of different types of agents, a carefully controlled series of mixes employing 13 air-entraining agents was prepared, and the void parameters were determined by the linear traverse method[1,8,12] (Fig. 2 and Table 1). In this series, all concretes contained ¾-in. maximum aggregate, and water-cement ratio, slump, and amount of compaction were held within close limits, the principal variable being the type and amount of agent required for an air content of about 5 percent as determined on the freshly mixed concrete by the air meter. Agents A, B, C, D, and J are well known air-entraining agents meeting the requirements of ASTM C 260. Agent I is not a true air-entraining

*Unpublished data of the Engineering Laboratories, Bureau of Reclamation, Denver, Colo.

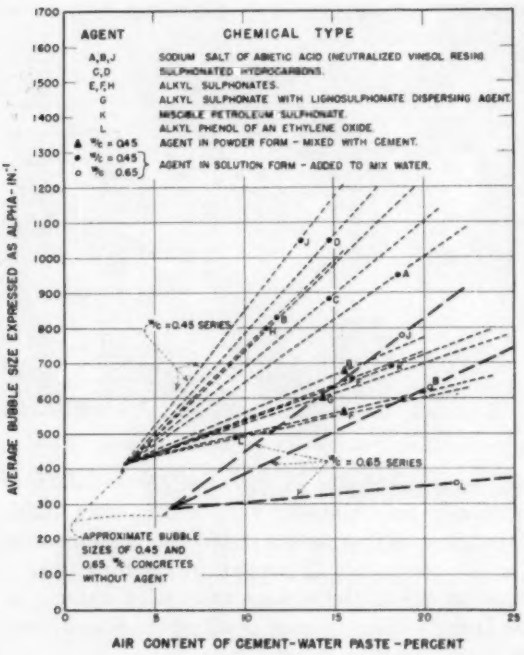
AGENT
CHEMICAL TYPE
A,B,J SODIUM SALT OF ABIETIC ACID (NEUTRALIZED VINSOL RESIN)
C,D SULPHONATED HYDROCARBONS.
E,F,H ALKYL SULPHONATES.
G ALKYL SULPHONATE WITH LIGNOSULPHONATE DISPERSING AGENT.
K MISCIBLE PETROLEUM SULPHONATE.
L ALKYL PHENOL OF AN ETHYLENE OXIDE.
▲ % = 0.45 AGENT IN POWDER FORM - MIXED WITH CEMENT.
● % = 0.45
○ % 0.65 AGENT IN SOLUTION FORM - ADDED TO MIX WATER.
% =0.45 SERIES
% = 0.65 SERIES
APPROXIMATE BUBBLE SIZES OF 0.45 AND 0.65 % CONCRETES WITHOUT AGENT
AVERAGE BUBBLE SIZE EXPRESSED AS ALPHA - IN.-1
AIR CONTENT OF CEMENT-WATER PASTE - PERCENT

Fig. 2—At the same air content in the paste, the average bubble size was influenced by the type and form of the agent and the amount of fines in the concrete, as expressed by the water-cement ratio

agent; it fails to effect entrainment of the desired amount of air, even if introduced in large amounts. Agent L is an air-entraining admixture manufactured outside the United States.

Effect on bubble size and spacing

Relative efficiency of the various agents, as indicated by the amount required to entrain a constant amount of air, varied widely as expected. However, significant differences in average bubble size and spacing factor (Table 1 and Fig. 2) also resulted from use of the different agents. In general, it appears that derivatives of abietic acid and sulfonated hydrocarbons produce somewhat smaller voids than do some other types of air-entraining agents. Also Agent B used in powder form produced slightly larger bubbles than when previously dissolved in a portion of the mixing water. Further tests may determine whether this is a characteristic phenomenon.

Although the freezing and thawing resistances of these particular concretes were excellent, except for concrete containing Agent L, there may be conditions, such as the use of a borderline aggregate,* under which this resistance may be more influenced by the type of agent and characteristics of the air void system produced by it. The divergence of the dashed lines of Fig. 2 indicates that the

*One which under the conditions of the freeze-thaw test[17] is considerably less resistant than the excellent aggregate from the vicinity of Coulee Dam used in this series.

differences in bubble *size* can be attributed to the particular type rather than to the amount of agent used.

Note that the concretes with the 0.45 water-cement ratio are divided into two general groups. Agents A, B, C, D, J, and H in liquid form produced small bubbles while the group containing Agents B (powder), E, F, G, and L produced much larger bubbles. The graphs show that this effect must be attributed to the agent itself and cannot be explained by differences in total air content. Three concretes of 0.65 water-cement ratio are included to illustrate that reducing the amount of fines in the mixture increases the average bubble size, but the effect of the *type* of agent on the bubble size remains evident.

TABLE 2—EFFECT OF TWO AIR-ENTRAINING ADMIXTURES ON VOID SIZE DISTRIBUTION IN CONCRETE*

Diameter of voids, microns	Number of voids per cu cm of concrete†	
	Agent D	Agent L
20	210,470	605
40	91,646	3403
60	30,761	2494
80	11,501	1608
100	7319	1023
120	4125	554
140	1056	548
160	1344	272
180	539	278
200	254	247
250	245	173
300	87.7	77.6
350	129	52.8
400	61.1	28.0
450	54.3	3.8
500	18.1	1.8
600	11.0	14.0
700	8.0	3.2
800	10.6	5.6
900	0.4	4.0
1000	1.5	4.6
2000	1.0	0.8
3000	1.5	0.2
4000	—	0.2
Total	359,645‡	11,401‡

*See Table 1 for mixture proportioning data.
†Calculated in accordance with the method established by Lord and Willis.[18]
‡Corresponding numbers of voids per cu in. of concrete are 5,894,581 and 186,862, respectively.

As mentioned, Agents A, B, C, D, and J produce voids of small average size, and the spacing factor is satisfactory (0.0040 to 0.0071 in.). The bubbles entrained by Agents E, F, G, H, I, and L are coarser and the spacing factor is comparatively large (0.0071 to 0.0140 in.).

Comparison between the void systems produced by the satisfactory agents represented by Agent D in this series and the inferior Agent L can be obtained by calculation based upon the modified linear traverse method, using the mathematical development devised by Lord and Willis[18] (Table 2). For example, the concrete containing Agent D entrained approximately 360,000 voids per cu cm and about 60 percent of the voids are less than 20 microns in diameter. On the other hand, with Agent L, the concrete contained about 11,400 voids per cu cm and only about 5 percent of these are less than 20 microns in diameter.

Void section frequency distributions produced by Agent J (α = 1050 in.$^{-1}$) and Agent L (α = 480 in.$^{-1}$) were determined by areal traverse* (Fig. 3). The higher values of α for the specimen containing Agent J result from the abundance of void sections less than 100 microns in diameter; also, approximately 50 percent of the void sections are less than 50 microns in diameter.

*No discrepancy exists between the calculated size distribution of *void* diameters indicated in Table 2 and the distribution of *void sections* indicated in Fig. 2. The curves are established directly from the diameters of void sections measured on a plane surface through the concrete. The probability that such a plane will intersect a given void decreases directly with the diameter of the void. Hence, the proportion of voids of given size indicated by the areal traverse will increase with the diameter of the void and the indicated proportion of small voids and of void sections of small diameter will be considerably less than the actual. Therefore, although the proportion of *void sections* less than 20 microns in diameter is indicated to be only 16 percent for the concrete containing Agent J, the actual proportion of *voids* less than 20 microns in diameter probably is close to 60 percent.

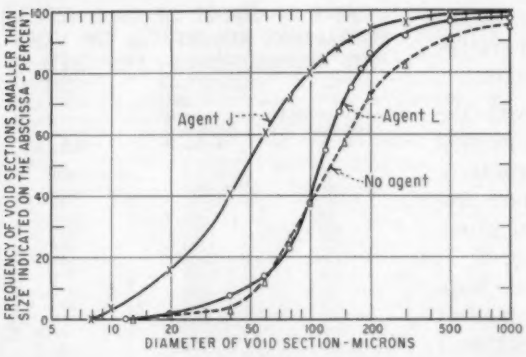

Fig. 3—Change in distribution of void-section size with use of satisfactory (Agent J) and inferior (Agent L) air-entraining admixtures

Conversely, in the specimens containing Agent L, void sections less than 100 microns in diameter are no more abundant than in non-air-entrained concrete, and void sections less than 50 microns in diameter constitute only 10 percent of the total number. This void system is characteristic of that of non-air-entrained concrete of moderate water-cement ratio. Although no void sections less than 13 microns in diameter were observed in this concrete, void sections less than 10 microns in diameter are not unusual in non-air-entrained concrete.

To determine the variation within the void system of concrete from place to place in specimens, another series of mixtures containing 1½-in. maximum-size aggregate was prepared and cast as 6 x 12-in. cylinders (Table 3), then sawed to provide 3 x 3 x 6-in. specimens. The void parameters obtained in this series compare favorably with the results obtained previously (Table 1). Here again, Agent L produced coarse bubbles and failed to retain the air indicated to be present immediately following mixing. The loss in air is attributed to the combination of: (1) a poor quality film produced by this agent, which offers but little resistance to migration of air from small to large voids, which in turn become more buoyant and therefore more susceptible to upward migration and final release into the atmosphere during placing and compacting; and (2) the agitation incident to handling and placing. Although the air content was found to be consistently higher in the top half of specimens than in the bottom half, the spacing factor was not affected significantly. Data obtained from an extensive investigation into the effects of vibration on various properties of air-entrained concrete verify these results, and in addition show specimens from the bottom half to be stronger and more durable than those from the top half of the original cylinders despite the negligible difference existing in the magnitude of the spacing factor. These data will be discussed later.

Effect on freezing and thawing resistance

The influence of type of agent, which was so significant on α and spacing factor $\bar{L}$, was not so readily apparent on freezing and thawing resistance. In only one instance, that of specimens containing Agent L, was resistance adversely affected. Agent L was particularly ineffective in retaining the

TABLE 3—EFFECT OF AIR-ENTRAINING ADMIXTURES ON THE VOID SYSTEM IN PORTIONS OF CONCRETE SPECIMENS*

Air-entraining agent	Quantity of air-entraining admixture per cu yd of concrete	Air content, percent by gravimetric method	Portion of specimen	Void parameters determined microscopically				
				Air content, percent	Chord intercept, in.	Specific surface, $in.^{-1}$	Voids intercepted per in.	Spacing factor, $\overline{L}$, in.
None	—	0.4	Top	1.68	0.0156	256	1.078	0.0293
			Bottom	0.62	0.0107	372	0.573	0.0309
			Average	1.15	0.0131	314	0.826	0.0301
B (solution)	1500 ml	6.0	Top	5.63	0.0045	889	12.520	0.0046
			Bottom	4.30	0.0048	833	9.003	0.0055
			Average	4.96	0.0046	861	10.762	0.0050
J	580 ml	5.0	Top	5.18	0.0041	976	12.647	0.0045
			Bottom	3.81	0.0039	1026	9.893	0.0049
			Average	4.49	0.0040	1001	11.270	0.0047
L	253 ml	5.2	Top	3.38	0.0085	470	3.984	0.0107
			Bottom	2.76	0.0078	513	3.544	0.0108
			Average	3.07	0.0082	492	3.764	0.0108

*1½-in. maximum-size natural aggregate; 34.2 percent sand in the aggregate; water-cement ratio = 0.53; 213 to 261 lb water per cu yd; slump = 2.6 to 4.6 in.; paste content = 20.15 to 24.70 percent; 6 x 12-in. cylinders, placed in two lifts, each lift vibrated 5 sec.

indicated initially entrained air or in developing a satisfactory system of voids ($\overline{L} = 0.0140$) in concrete having a water-cement ratio of 0.45. These deficiencies were reflected in the relatively low resistance developed. On the other hand, a similar mixture except for water-cement ratio of 0.65 contained more air in the hardened concrete than was indicated in the fresh concrete, possessed a better spacing factor (0.010 in.) than its richer companion, but had low resistance to freezing and thawing. Failure in this case consisted of a rapid expansion in combination with a heavy scaling neither consistent with nor anticipated from the magnitude of $\overline{L}$, and is probably due to the excessive size of the air voids and their sporadic and nonuniform distribution throughout the concrete. Concretes containing the other agents consistently possessed much better void systems and all developed very little, if any, expansion during the freezing and thawing test, a result found to be typical of air-entrained concrete containing the superior basaltic aggregate from the vicinity of Grand Coulee Dam. Had these concretes contained an aggregate of average or poor quality in a leaner mix, the effect of type of air-entraining agent on freezing and thawing resistance would probably have been more evident.

PROPORTION OF AIR-ENTRAINING AGENT

If other factors are held constant, an increase in the air content of cement paste of given water-cement ratio will increase the interface area through which diffusion takes place and decrease the distance through which the dissolved air must diffuse. Consequently, the rate of dissolution of small bubbles and growth of larger bubbles will increase. Moreover, if the film between bubbles becomes too thin, coalescence of bubbles decreases their number and increases their average size. Hence, α will tend to decrease and $\overline{L}$ to increase at a greater rate prior to hardening of the concrete if the air content is increased.

However, if the greater air content is developed by use of an increased proportion of air-entraining admixture, within the range of concentration

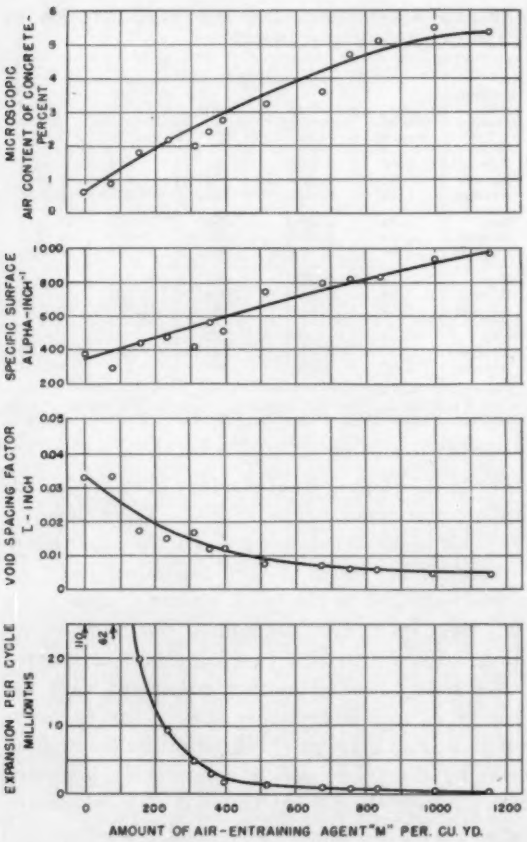
MICROSCOPIC AIR CONTENT OF CONCRETE-PERCENT
SPECIFIC SURFACE ALPHA-INCH^{-1}
VOID SPACING FACTOR L - INCH
EXPANSION PER CYCLE MILLIONTHS
AMOUNT OF AIR-ENTRAINING AGENT "M" PER. CU. YD.

Fig. 4—Effect of varying amounts of air-entraining agent on parameters of void system and freezing and thawing resistance

ordinarily employed, the decrease in surface tension and increased thickness of the adsorbed layer at the air-water interface in part will offset the tendency toward an increased rate of diffusion of air. Also, the reduction in surface tension permits more breakdown of large bubbles into small ones on vigorous agitation, and the rate of coalescence of bubbles will be decreased. Transmission of air from bubble to bubble also will be decreased to the extent that a thicker and more concentrated film of adsorbed polar molecules at the air-water interface reduces passage of air out of or into the bubbles.

Data presented below will demonstrate that in the range of practical air content of concrete the effect of increased air-entraining admixture outweighs the effect of increased air content and so results in increased α and decreased $\bar{L}$. With addition of excessive proportions of air-entraining admixture, decrease in the difference between the concentration of admixture in the film and in the bulk water phase will reduce the elasticity of the adsorbed film sufficiently to permit more ready coalescence of bubbles so that α will tend to decrease.

TABLE 4—EFFECT OF PROPORTION OF AIR-ENTRAINING ADMIXTURE

Proportion of air-entraining admixture (M), ml per cu yd	Water-cement ratio by weight	Water content, lb per cu yd	Paste content, percent	Void parameters determined microscopically					Freeze-thaw expansion per cycle, millionths
				Air content, percent	Chord intercept, in.	Specific surface, $in.^{-1}$	Voids intercepted per in.	Spacing factor, $\bar{L}$, in.	
0	0.58	299	27.5	0.6	0.0103	388	0.58	0.036	110
80	0.57	295	27.3	0.9	0.0140	286	0.64	0.036	62
160	0.56	288	26.9	1.8	0.0092	435	1.96	0.0172	20
240	0.55	283	26.6	2.2	0.0085	471	2.59	0.0148	9
320	0.53	277	26.3	2.0	0.0095	420	2.10	0.0169	5
360	0.53	274	26.1	2.4	0.0072	556	3.34	0.0121	3
400	0.52	272	26.0	2.7	0.0080	500	3.38	0.0124	2.1
520	0.51	264	25.4	3.2	0.0054	741	5.93	0.0077	1.5
680	0.49	251	24.7	3.6	0.0051	784	7.06	0.0069	1.1
760	0.48	248	24.5	4.7	0.0049	816	9.59	0.0057	0.9
840	0.47	244	24.3	5.1	0.0048	826	10.53	0.0054	0.8
1000	0.46	239	24.0	5.5	0.0043	937	12.88	0.0046	0.6
1160	0.44	234	23.7	5.4	0.0041	976	13.18	0.0044	0.5

Effect on bubble size and spacing

To study the relationship of α and $\bar{L}$ to the amount of air-entraining admixture, a series of 3 x 6-in. cylinders was prepared from similar mixtures containing ¾-in. maximum-size aggregate and constant cement content, to which increasing amounts of Agent M were added to effect small increases in air content (Table 4). Water content was varied, resulting in water-cement ratios ranging from 0.45 to 0.58, to provide approximately the same slump. Each cylinder was vibrated approximately 5 sec. The air content after vibration ranged from 0.6 to 6.2 percent, as the amount of admixture was increased from 0 to 1160 ml per cu yd. Thirteen specimens, including one without any air-entraining admixture, were analyzed microscopically (Table 4 and Fig. 4). The data show that within the range investigated, the use of additional amounts of air-entraining admixture results in increased air content and α (the average bubble *size* decreases), in decreased $\bar{L}$, and in increased resistance to freezing and thawing. Values of α varied from less than 600 $in.^{-1}$ at 3 percent air to approximately 1000 $in.^{-1}$ at 6 percent air. As the air content approaches 6 percent, α appears to increase only slightly. Other tests indicate continued slight increases in α with increases in air content to as much as 12 percent. $\bar{L}$ decreased progressively with increases in α and freezing and thawing resistance increased. The effects on α described in this section have been attributed entirely to variations in amount of air-entraining admixture. However, it should be emphasized that changes in water-cement ratio of concretes containing either a constant amount of air or a constant amount of admixture also have significant effects on α. These effects will be evaluated and discussed later but, in this series, because of the extremely small differences in water-cement ratio when compared with the differences in the amount of agent used, they are believed to be of secondary importance.

Variation in the proportion of air-entraining admixture used is also reflected in the distribution of void sizes. Data are available for a series of concretes containing varying proportions of Agent J (Table 5 and Fig. 5). For the concrete whose air content is 8.4 percent, α is 1140 $in.^{-1}$ and approximately 65 percent of the void sections are less than 50 microns in diameter.

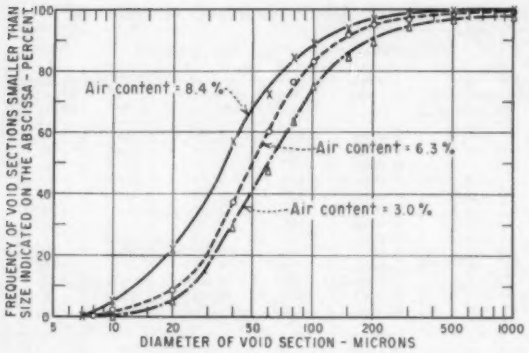

Fig. 5—Change in distribution of void-section size with change of air content induced by varying the proportion of air-entraining admixture

For the concrete whose air content is 3.0 percent, α is 625 in.$^{-1}$ and only about 40 percent of the void sections are less than 50 microns in diameter. Appreciable differences occur also in the abundance of void sections less than 10 microns in diameter.

In supplementary tests the effect of the proportion of air-entraining admixture on the value of α for hand-rodded and for vibrated concretes was found to be similar. This is demonstrated by a concrete taken directly from the mixer and neither formed nor compacted; after hardening the air content was 13.5 percent and α = 702 in.$^{-1}$ After placing and compacting some of the same concrete, the air content of the hardened concrete was reduced to 6.5 percent and α was increased to 1250 in.$^{-1}$ Nevertheless, and despite the violent agitation attendant to the placing operations, $\overline{L}$ in the hardened concrete increased only from 0.0024 to 0.0031 in. From this, it may be concluded that with a given proportion of air-entraining admixture, if other factors are held essentially constant, concrete develops a void system such that the spacing factor $\overline{L}$ varies only slightly, regardless of manipulation of the fresh concrete, within practical limits, and in spite of large changes in α and air content.

Optimum air content

It is possible to evaluate the concept of optimum air content in the light of these data. As generally conceived, optimum air content is that air content which yields a practical maximum resistance to freezing and thawing with minimum loss of strength. As we have seen above, a given proportion of a satisfactory air-entraining admixture, sufficient to entrain as much or more than the amount of air recommended by ACI Committee 613, induces a system of air voids characterized by a spacing factor which can be modified only slightly by manipulation of the plastic concrete. Since the resistance to freezing and thawing of cement paste of moderate water-cement ratio depends primarily upon the spacing factor, we may conclude that the optimum air content for any given concrete is that air content obtained with the applied method of compaction when the concrete removed from the mixer possesses,

TABLE 5—EFFECT OF PROPORTION OF AIR-ENTRAINING ADMIXTURE ON THE VOID SYSTEM OF CONCRETE HAVING CONSTANT W/C RATIO AND SLUMP*

Proportion of air-entraining admixture ml per cu yd†	Water content, lb per cu yd	Sand content, percent of aggregate	Slump, in.	Paste content, percent	Void parameters determined microscopically				
					Air content, percent	Chord intercept, in.	Specific surface, $in.^{-1}$	Voids intercepted per in.	Spacing factor, $\bar{L}$, in.
302	303	48	2.5	29.5	3.0	0.0064	625	4.65	0.0101
587	278	45	2.9	26.8	6.3	0.0051	784	12.36	0.0054
1,509	251	42	2.7	24.1	8.4	0.0035	1140	24.14	0.0025

*1½-in. maximum-size natural aggregate; 42 to 48 percent sand in aggregate; water-cement ratio = 0.50; 6 x 12-in. cylinders, placed in two lifts, each lift vibrated 2 sec.
†Agent J

after hardening, adequate strength and spacing factor consistent with the desired resistance to freezing and thawing. The magnitude of the spacing factor, and hence the magnitude of the air content at the optimum, depends upon the freezing and thawing procedure applied or used as reference. It follows that the determined magnitude of the optimum air content of any given concrete under a given set of conditions may vary widely, depending upon the method and intensity of compaction, but the optimum air content, whatever it may be in magnitude, should be obtained with the same amount of a given air-entraining admixture.

Freezing and thawing resistance

Fig. 4, a plot of data in Table 4, shows the correlation between the amount of air-entaining agent, and the parameters of the void system and the resistance of fog cured concrete to freezing and thawing. Direct or inverse relationships are indicated in all cases. Fig. 6 shows the correlation between L and the freezing and thawing resistance of the same concretes. These results agree remarkably with previous findings,[19,20,21] whose average is also plotted in Fig. 6, when it is recalled that results from different concretes, different methods of specimen surface preparation, and different test procedures were used to get the average. Another factor contributing to the displacement from the average of the curve representing concretes containing Agent M is the presence of a dispersing agent in Agent M which, in increasing amounts, significantly reduces water-cement ratio. Hyperbolic shape of the curves shows that progressively smaller spacing factors provide, within limits, increasingly greater freezing and thawing resistance with increasing margin of safety.

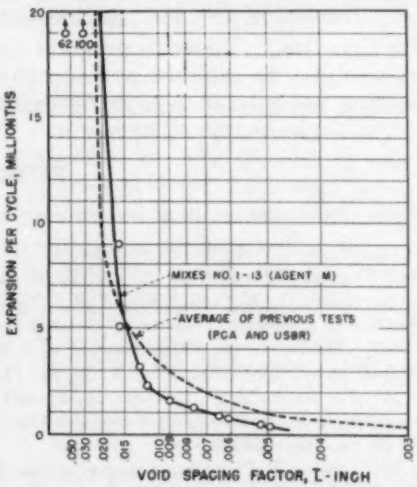

Fig. 6—Correlation of void spacing factor and freezing and thawing expansion. Amount of agent varied. Dashed curve from Reference 21

CONCLUSIONS

1. The resistance to freezing and thawing of moist cured, air-entrained concrete, as measured by expansion per cycle is influenced to a great degree by the magnitude of the void spacing factor[12,17] with which it varies inversely, and which in turn is determined by size distribution and frequency of air voids. With but few isolated exceptions, those variables which tend to decrease spacing factor also tend to increase freezing-thawing resistance.

2. The size distribution and frequency of air voids and concomitantly the spacing factor and freezing and thawing resistance, are influenced by many factors, among the most important of which are the kind and the amount of air-entraining agent. The average size of the voids and the computed spacing factor tend to decrease (1) with increasing proportion of air-entraining agent, at constant water-cement ratio, and (2) at nearly constant air content, with decreasing water-cement ratio within the range of workable mixtures.

3. Spacing factor appears to be an inherent characteristic of each concrete mixture, being determined by the physical and physical-chemical properties of the constituents and the proportion and efficiency of the air-entraining agent. The spacing factor is not altered materially by manipulation incidental to handling, placing, and compaction, although the air content and α may vary widely.

4. The data indicate that a satisfactory entrained air void system is assured if "Recommended Practice for Selecting Proportions for Concrete (ACI 613-54)" is followed, provided the air-entraining admixture will meet the requirements of ASTM C 260 "Specifications for Air-Entraining Admixtures for Concrete." Under these conditions, the specific surface α of the air void system may be expected to range from 600 to 1000 in.$^{-1}$ and the computed spacing factor from about 0.008 to 0.004 in. respectively.

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Note: References numbered consecutively beginning with Part 1 of this four part series.

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Design of Symmetrical Columns with Small Eccentricities in One or Two Directions*

By FREDERICK P. WIESINGER†

SYNOPSIS

A comprehensive and general method is presented for the design of columns according to the ACI Code, where e'/t is not more than $\frac{3}{8}$ in either direction. Tables to speed up time-consuming trial and error computations for all standard cases and a simple method for the construction of straight-line charts are included.

NOTATION

The notation is substantially the same as that of "Building Code Requirements for Reinforced Concrete (ACI 318-56)" except that the eccentricity of the resultant load is designated as e', the least lateral dimension of the column as t', and that the stresses are used in kips per sq in. Special notation is explained in the text.

A_g = over-all (gross) area, sq in.
A_s = cross-sectional area of reinforcing steel, sq in.
α_g = shape factor [see Eq. (1)]
α_s = pattern factor [See Eq. (2)]
e' = eccentricity of the resultant load on the column, measured from the gravity axis, in.
F_a = nominal allowable axial unit stress ($0.225 f_c' + f_s p_g$), kips per sq in.
F_b = allowable bending unit stress, kips per sq in., that would be permitted if bending stress only existed = $0.45 f_c'$
f_a = nominal axial unit stress = N/A_g, kips per sq in.
f_b = bending unit stress (actual) = Ne'/S, kips per sq in.
f_c = actual concrete stress, kips per sq in.
f_s = nominal allowable stress in vertical column reinforcement: 16 kips per sq in. for intermediate grade steel and 20 kips per sq in. for rail or hard grade steel [See Section 1103(a), ACI 318-56]
gt = center-to-center dimension of extreme steel in the direction of bending, in.
h = unsupported length of column, in. (See Section 1102, ACI 318-56)
I = moment of inertia of column in the direction of bending, in.^{4}
I_g = moment of inertia of over-all or gross area in the direction of bending, in.^{4}
I_s = moment of inertia of reinforcing steel area in the direction of bending, in.^{4}
N = axial load applied to column, kips
P = equivalent axial load, kips
p_g = A_s/A_g
Q = reduction factor. See Eq. (4)
S = section modulus of column (of transformed area) in the direction of bending, in.^{3}
t = out-to-out dimension of column in the direction of bending, in.
t' = least lateral dimension of column, in.

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TABLE 1—VALUES OF α_g

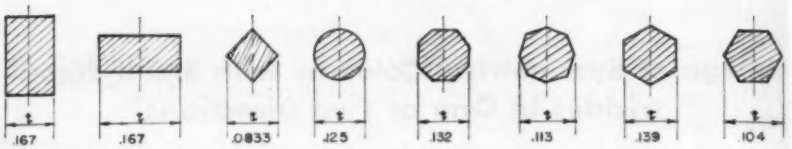
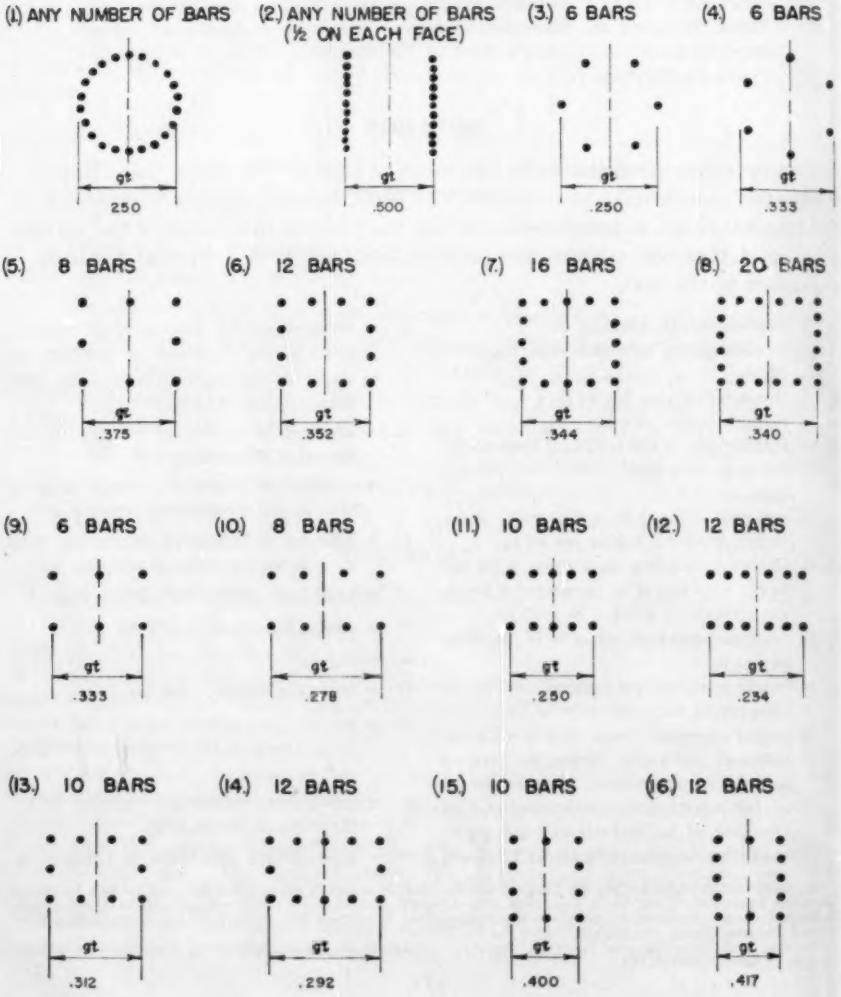


TABLE 2—VALUES OF α_s WITH ALL BARS OF SAME SIZE, BAR SPACING CONSTANT ALONG FACE



DERIVATION OF FORMULAE

To design a column for a given load one has to determine both the size of the column and the amount of reinforcement, assuming that allowable stresses, shape of column, and pattern of reinforcement are predetermined.

A basic equation giving the required column size A_g will be developed. To do this it is necessary to express the section modulus of the member as a function of A_g. This can be done by using the radius of gyration of the section; nevertheless, it will be more convenient to introduce two new, dimensionless variables: "shape factor" for the gross area, designated as α_g and "pattern factor" for the steel, designated as α_s.

$$\alpha_g = \frac{2\,I_g}{t^2\,A_g} \qquad (1)$$

$$\alpha_s = \frac{2\,I_s}{(gt)^2\,A_s} \qquad (2)$$

It can be seen readily that these will be independent of sizes and represent only the shape and the pattern. (Example 4 gives an illustration.) Tables 1 and 2 include most every common case.

The moment of inertia of the whole section can be expressed as:

$$I = I_g + (n-1)\,I_s$$

Expressing I_g and I_s from Eq. (1) and (2) and substituting:

$$I = \tfrac{1}{2}\alpha_g t^2 A_g + \tfrac{1}{2}\alpha_s (n-1)\,g^2 t^2 A_s$$

The section modulus of the member becomes:

$$S = \frac{I}{t/2} = t\,[\alpha_g A_g + \alpha_s (n-1) g^2 A_s] \qquad (3)$$

or, since $A_s = p_g A_g$

$$S = tA_g[\alpha_g + \alpha_s\,(n-1) g^2 p_g] \qquad (3a)$$

The equation governing the design of columns is given in Section 1109 (a) of ACI 318-56:

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} \leqq 1$$

Substituting the quantities defined in Section 1100:

$$\frac{N/A_g}{0.225\,f_c' + f_s p_g} + \frac{Ne'/S}{F_b} \leqq 1$$

TABLE 3—VALUES OF Q [0.225 f_c' + $f_s p_g$]*

f_s, kips per sq in.	f_c', kips per sq in.	Tied columns (Q = 0.8)				Spirally reinforced columns (Q = 1.0)			
		p_g = 0.01	p_g = 0.02	p_g = 0.03	p_g = 0.04	p_g = 0.01	p_g = 0.02	p_g = 0.03	p_g = 0.04
16	2	0.488	0.616	0.744	0.872	0.610	0.770	0.930	1.090
	2.5	0.577	0.705	0.834	0.961	0.722	0.882	1.042	1.202
	3	0.668	0.796	0.924	1.052	0.835	0.995	1.155	1.315
	3.75	0.803	0.931	1.059	1.187	1.004	1.164	1.324	1.484
	5	1.028	1.156	1.284	1.412	1.285	1.445	1.605	1.765
20	2	0.520	0.680	0.840	1.000	0.650	0.850	1.050	1.250
	2.5	0.610	0.770	0.930	1.090	0.762	0.962	1.162	1.362
	3	0.700	0.860	1.020	1.180	0.875	1.075	1.275	1.475
	3.75	0.835	0.995	1.155	1.315	1.044	1.244	1.444	1.644
	5	1.060	1.220	1.380	1.540	1.325	1.525	1.725	1.925

*For long columns multiply the tabulated values by (1.3 − 0.03 h/t')

In order to account in an orderly fashion for the reduction for tied columns [Section 1104(a)] and the reduction for long columns (Section 1107), a "reduction factor" designated as Q will be introduced.

$$\left.\begin{array}{ll} Q = 1.0 & \text{for short spiral columns} \\ Q = 0.8 & \text{for short tied columns} \\ Q = 1.3 - 0.03\,h/t' & \text{for long spiral columns} \\ Q = 0.8\,(1.3 - 0.03\,h/t') & \text{for long tied columns} \end{array}\right\} \quad \ldots\ldots\ldots\ldots (4)$$

Substituting the value of S given in Eq. (3a) and including Q, the above expression becomes

$$\frac{N/A_g}{Q\,[0.225\,f_c' + f_s p_g]} + \frac{(N/A_g)\,(e'/t)}{F_b\,[\alpha_g + \alpha_s\,(n-1)g^2 p_g]} \leqq 1 \quad \ldots\ldots\ldots\ldots\ldots (5)$$

or

$$\frac{N}{Q\,[0.225\,f_c'\,A_g + f_s A_s]} + \frac{Ne'/t}{F_b\,[\alpha_g A_g + \alpha_s\,(n-1)g^2 A_s]} \leqq 1 \quad \ldots\ldots\ldots (5a)$$

Eq. (5) can be rearranged to yield an expression for A_g:

$$\text{required } A_g = \frac{N}{Q\,[0.225\,f_c' + f_s p_g]} + \frac{Ne'/t}{F_b\,[\alpha_g + \alpha_s\,(n-1)g^2 p_g]} \quad \ldots\ldots\ldots (6)$$

The denominators of this equation for the most common cases are given in Tables 3 and 4. The 33⅓ percent increase of the allowable stresses given in Section 603(c) of the ACI Code can be achieved by multiplying the tabulated values in both tables by 1.333.

For bending in both directions the second term will have to be used separately for each axis according to Section 1109(b). See Example 4.

In order to compute the required reinforcement, an expression for p_g can be developed. However, this expression is cumbersome even for bending in

one direction only. An easier method for determining p_g will be shown in Examples 1 and 4, and a solution with a chart is given in Example 2.

It might be desirable in some cases to compute the capacity of a given column section for a given eccentricity. Eq. (5a) can be rearranged to yield:

$$\text{allowable } N = \frac{1}{\dfrac{1}{Q\,[0.225\,f_c'A_g + f_sA_s]} + \dfrac{e'/t}{F_b\,[\alpha_gA_g + \alpha_s\,(n-1)g^2A_s]}} \quad \ldots\ldots (7)$$

See Example 3.

APPLICATIONS OF THE METHOD

Example 1

Using the tables, design a tied column for $N = 300$ kips, $e' = 6$ in., $f_c' = 3$ kips per sq in., $f_s = 16$ kips per sq in. (intermediate grade steel). (Example from article by Savran in May 1957 ACI JOURNAL, *Proc.* V. 53, p. 1081).

Assume $p_g = 0.02$ and $t = 20$ in.($gt = 15$ in., $g = 15/20 = 0.75$). By Eq. (6),

$$\text{required } A_g = \frac{N}{Q\,[0.225\,f_c' + f_sp_g]} + \frac{Ne'/t}{F_b\,[\alpha_g + \alpha_s\,(n-1)g^2p_g]}$$

Find denominators from Table 3 and with interpolation from Table 4.

$$\text{required } A_g = \frac{300}{0.796} + \frac{1800/20}{0.293} = 377 + 307 = 684 \text{ sq in.}$$

The required width would be about 34 in. ($34 \times 20 = 680$ sq in.) Not good.

Try $t = 24$ in. ($g = 0.79$)

$$\text{If } p_g = 0.03, \text{ required } A_g = \frac{300}{0.924} + \frac{1800/24}{0.338} = 325 + 222 = 547 \text{ sq in.}$$

$$\text{If } p_g = 0.04, \text{ required } A_g = \frac{300}{1.052} + \frac{1800/24}{0.377} = 285 + 199 = 484 \text{ sq in.}$$

Use 22 x 24 in., $A_g = 528$ sq in.

Find required p_g by interpolating between $p_g = 0.03$ for $A_g = 547$ and $p_g = 0.04$ for $A_g = 484$:

$$p_g = 0.03 + (0.04 - 0.03)\,\frac{547 - 528}{547 - 484} = 0.033$$

(A straight-line interpolation is theoretically not correct; nevertheless, the result will be sufficiently accurate for all practical purposes.)

$$\text{required } A_s = 0.033 \times 528 = 17.4 \text{ sq in.}$$

TABLE 4—VALUES OF F_b $[\alpha_g + \alpha_s(n-1)g^2p_g]$; $F_b = 0.45\ f_c'$; $n = 30/f_c'$

f_c', kips per sq in.	g	Rectangular section, one-half of bars on each face $\alpha_g = 0.1667$ $\alpha_s = 0.5000$				Rectangular section, spirally reinforced $\alpha_g = 0.1667$ $\alpha_s = 0.2500$				Circular section $\alpha_g = 0.1250$ $\alpha_s = 0.2500$			
		$p_g = 0.01$	$p_g = 0.02$	$p_g = 0.03$	$p_g = 0.04$	$p_g = 0.01$	$p_g = 0.02$	$p_g = 0.03$	$p_g = 0.04$	$p_g = 0.01$	$p_g = 0.02$	$p_g = 0.03$	$p_g = 0.04$
2	.9	0.201	0.252	0.304	0.354	0.176	0.201	0.227	0.252	0.138	0.164	0.189	0.214
	.8	0.190	0.230	0.271	0.312	0.170	0.190	0.211	0.230	0.132	0.153	0.173	0.194
	.7	0.181	0.212	0.243	0.274	0.166	0.181	0.196	0.222	0.127	0.143	0.158	0.175
2.5	.9	0.237	0.288	0.337	0.388	0.213	0.237	0.262	0.288	0.165	0.191	0.216	0.240
	.8	0.226	0.267	0.306	0.345	0.207	0.227	0.246	0.267	0.160	0.180	0.200	0.220
	.7	0.218	0.248	0.279	0.308	0.203	0.218	0.233	0.249	0.150	0.171	0.186	0.202
3	.9	0.274	0.324	0.372	0.421	0.250	0.274	0.298	0.324	0.193	0.217	0.243	0.267
	.8	0.263	0.302	0.341	0.381	0.244	0.263	0.283	0.302	0.187	0.208	0.227	0.247
	.7	0.255	0.285	0.314	0.344	0.240	0.255	0.270	0.285	0.183	0.198	0.213	0.228
3.75	.9	0.329	0.376	0.425	0.472	0.305	0.329	0.352	0.376	0.234	0.258	0.283	0.307
	.8	0.319	0.356	0.395	0.432	0.300	0.319	0.338	0.356	0.229	0.248	0.268	0.286
	.7	0.310	0.339	0.368	0.396	0.295	0.310	0.324	0.339	0.226	0.240	0.254	0.268
5	.9	0.420	0.466	0.512	0.557	0.398	0.420	0.443	0.446	0.304	0.327	0.350	0.372
	.8	0.411	0.447	0.484	0.520	0.394	0.411	0.429	0.448	0.299	0.317	0.335	0.353
	.7	0.402	0.430	0.458	0.485	0.389	0.402	0.416	0.430	0.295	0.309	0.323	0.336

f_c', kips per sq in.	g	Rectangular section, bars as in Case 5, Table 2 $\alpha_g = 0.1667$ $\alpha_s = 0.3750$				Rectangular section, bars as in Case 6, Table 2 $\alpha_g = 0.1667$ $\alpha_s = 0.3520$				Rectangular section, bars as in Case 9, Table 2 $\alpha_g = 0.1667$ $\alpha_s = 0.3333$			
		$p_g = 0.01$	$p_g = 0.02$	$p_g = 0.03$	$p_g = 0.04$	$p_g = 0.01$	$p_g = 0.02$	$p_g = 0.03$	$p_g = 0.04$	$p_g = 0.01$	$p_g = 0.02$	$p_g = 0.03$	$p_g = 0.04$
2	.9	0.188	0.227	0.265	0.303	0.186	0.222	0.258	0.294	0.184	0.218	0.252	0.286
	.8	0.180	0.210	0.241	0.271	0.178	0.207	0.235	0.263	0.177	0.204	0.231	0.258
	.7	0.173	0.196	0.219	0.243	0.172	0.193	0.215	0.237	0.171	0.191	0.212	0.232
2.5	.9	0.225	0.263	0.300	0.338	0.223	0.258	0.293	0.329	0.221	0.254	0.288	0.321
	.8	0.217	0.247	0.277	0.306	0.215	0.243	0.271	0.299	0.214	0.240	0.267	0.293
	.7	0.210	0.233	0.256	0.278	0.209	0.230	0.251	0.273	0.208	0.228	0.248	0.268
3	.9	0.262	0.299	0.336	0.373	0.260	0.294	0.329	0.364	0.258	0.291	0.323	0.356
	.8	0.254	0.283	0.312	0.342	0.252	0.280	0.307	0.334	0.251	0.277	0.303	0.329
	.7	0.245	0.270	0.292	0.314	0.246	0.267	0.288	0.309	0.245	0.265	0.285	0.304
3.75	.9	0.317	0.353	0.389	0.425	0.315	0.349	0.382	0.416	0.313	0.345	0.377	0.409
	.8	0.310	0.338	0.366	0.395	0.308	0.334	0.361	0.388	0.306	0.332	0.357	0.382
	.7	0.303	0.325	0.346	0.368	0.302	0.322	0.342	0.363	0.301	0.320	0.339	0.358
5	.9	0.409	0.443	0.478	0.512	0.407	0.439	0.471	0.503	0.405	0.436	0.466	0.496
	.8	0.402	0.429	0.456	0.483	0.400	0.426	0.451	0.476	0.399	0.423	0.447	0.471
	.7	0.396	0.416	0.437	0.458	0.394	0.414	0.433	0.453	0.393	0.412	0.430	0.448

The actual selection of bars can be checked with Eq. (5a). If ten #11 bars are used, $A_s = 15.6$:

$$\frac{300}{0.8\,[0.675 \times 528 + 16 \times 15.6]} + \frac{1800/24}{1.35\,[0.1667 \times 528 + 5 \times 9 \times 0.79^2 \times 15.6]} =$$

$$0.617 + 0.422 = 1.039 > 1$$

Therefore the section is somewhat overstressed.

Example 2

Construct a chart (Fig. 1) for solving the problem of Example 1. (Use this method if several columns of the same type have to be designed for various combinations of N and e'.) Bending about one axis only. (For derivation see Appendix.) Column size = 22 x 24 in.; A_g = 528 sq in.

Mark on the vertical scale various values of $Q\,[0.225\,f_c' + f_s p_g]$. See Table 3; for $p_g = 0.01$ the value is 0.668; for $p_g = 0.02$ it is 0.796, etc. Also mark on the horizontal scale various values of $F_b\,[\alpha_g + \alpha_s (n-1) g^2 p_g]$. See Table 4. Connecting the corresponding points completes the chart. Enter on the vertical scale with $N/A_g = 300/528 = 0.568$ and on the horizontal scale with $(N/A_g)\,(e/t) = 0.568 \times 6/24 = 0.142$ and read required $p_g = 0.033$.

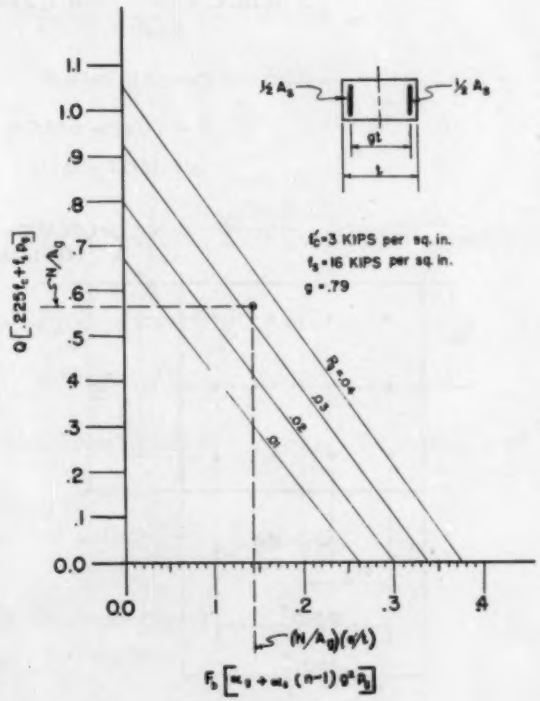


Fig. 1—Chart for solution of Example 2

Example 3

Determine the capacity of a 20 × 20-in. tied column if $f_c' = 3$ kips per sq in., $f_s = 20$ kips per sq in., $A_s = 4.74$ sq in., and $e' = 3$ in. (Example from *Concrete Reinforcing Steel Institute Design Handbook*, pp. 276, 277, and 278) Use Eq. (7):

$$N = \frac{1}{\frac{1}{0.8\,(0.675 \times 400 + 20 \times 4.74)} + \frac{3/20}{1.35\,(0.1667 \times 400 + 0.5 \times 9 \times 0.5625 \times 4.74)}}$$

$$= \frac{1}{0.00343 + 0.001415} = 206 \text{ kips}$$

Example 4

Determine the size of the bars to be used for the column shown in Fig. 2.

$f_c' = 4$ kips per sq in. ($F_b = 0.45 \times 4 = 1.8$ kips per sq in., $n = 30/4 = 7.5$), $f_s = 20$ kips per sq in., $N = 93$ kips. This example will be worked without the use of the tables in order to illustrate the method of obtaining the tabulated values.

About the y-axis:

$$\alpha_y = \frac{2 \times 18 \times 21^3/12}{18 \times 21 \times 21^3} = 0.1667 \text{ (See Table 1)}$$

Six of the ten bars are at $gt/2$ and four of them at $gt/6$:

$$\alpha_x = \frac{2[\,(6/10)A_s(gt/2)^2 + (4/10)A_s\,(gt/6)^2]}{A_s(gt)^2}$$

$$= 0.322 \text{ (See Case 13 in Table 2)}$$

$$e'/t = 13/21 = 0.62 < \tfrac{3}{5}$$

$$g = 15/21 = 0.715$$

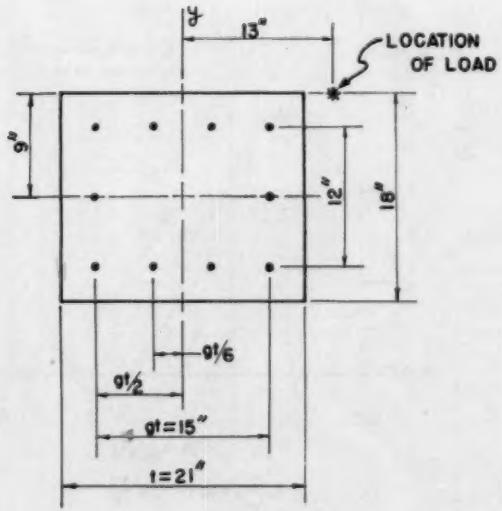


Fig. 2—Column used in Example 4

About the x-axis:

$$\alpha_g = 0.1667$$

$$\alpha_c = \frac{2[(8/10)A_s(gt/2)^2]}{A_s(gt)^2}$$

$$= 0.400 \text{ (See Case 15 in Table 2)}$$

$$e'/t = 9/18 = 0.5 < \tfrac{2}{3}$$

$$g = 12/18 = 0.667$$

If $p_g = 0.02$, from Eq. (6), by using the second term separately for each axis, find required gross area:

$$\text{required } A_g = \frac{93}{0.8(0.90 + 20 \times 0.02)} + \frac{93 \times 0.62}{1.80(0.1667 + 0.322 \times 6.5 \times 0.715^3 \times 0.02)}$$

$$+ \frac{93 \times 0.5}{1.80(0.1667 + 0.400 \times 6.5 \times 0.667^2 \times 0.02)}$$

$$= 89 + 170 + 136 = 395 \text{ sq in.}$$

If $p_g = 0.03$

$$\text{required } A_g = \frac{93}{0.8(0.90 + 20 \times 0.03)} + \frac{93 \times 0.62}{1.80(0.1667 + 0.322 \times 6.5 \times 0.715^3 \times 0.03)}$$

$$+ \frac{93 \times 0.5}{1.80(0.1667 + 0.400 \times 6.5 \times 0.667^2 \times 0.03)}$$

$$= 78 + 161 + 128 = 367 \text{ sq in.}$$

Get p_g for $A_g = 21 \times 18 = 378$ by interpolating between $A_g = 395$ for $p_g = 0.02$, and $A_g = 367$ for $p_g = 0.03$:

$$p_g = 0.02 + (0.03 - 0.02)\frac{395 - 378}{395 - 367} = 0.0261$$

$$A_s = 378 \times 0.0261 = 9.9 \text{ sq in.}$$

Use ten #9 bars (10.00 sq in.)

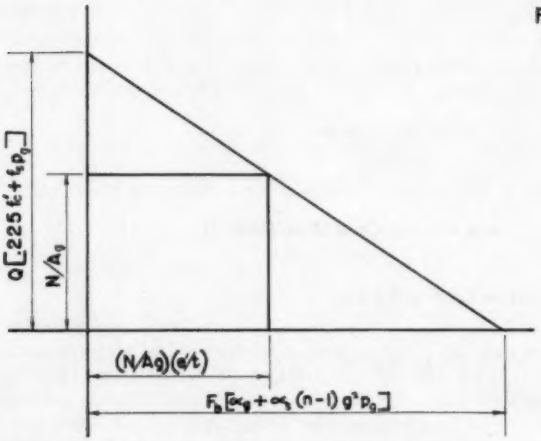


Fig. 3—Interaction diagram N/A_g and $N/A_g \times e'/t$

APPENDIX

Derivation of chart used in Example 2

From similar triangles, Fig. 3,

$$\frac{(N/A_g)\,(e'/t)}{F_b[\alpha_g + \alpha_s\,(n-1)g^2p_g]} = \frac{Q\,[0.225\,f_c' + f_sp_g] - N/A_g}{Q\,[0.225\,f_c' + f_sp_g]}$$

Rearranging yields Eq. (5).

Actual stresses

If desired, the actual concrete stress f_c can be determined from:

$$\frac{\text{axial load}}{\text{transformed area}} \pm \frac{\text{moment}}{\text{section modulus}}$$

See Eq. (3), again using the second term separately for each axis if bending in both directions exists. Thus:

$$f_c = \frac{N}{A_g + (n-1)A_s} \pm \frac{Ne'/t}{\alpha_gA_g + \alpha_s\,(n-1)g^2A_s} \qquad (8)$$

$$= \frac{N}{A_g}\left(\frac{1}{1 + (n-1)p_g} \pm \frac{e'/t}{\alpha_g + \alpha_s(n-1)g^2p_g}\right) \qquad (9)$$

Equivalent axial load with the use of coefficient B

The Code (ACI 318-56) provides in Section 1109(c) for design using an equivalent axial load P. This paper can be used to get an exact value for P with very little work, but this is not recommended because it leads to trial and error, much of which can be avoided by using the methods outlined above. However, it might be of interest to show how P can be obtained.

Introducing the reduction factor Q and using the notation of this paper, the equation from the Code can be given as:

$$P = (N/Q)\,[1 + (Be'/t)] \quad \ldots\ldots (10)$$

The Be'/t quantity has to used separately for each axis if bending in both directions exists.

The coefficient B is the same as the quantity $C \times D$ from the old Code (ACI 318-51) and an exact value can be obtained by computing:

$$C = \frac{Q\,[0.225\,f_c' + f_s p_g]}{F_b\,[1 + (n-1)p_g]} \quad \ldots\ldots (11)$$

and

$$D = \frac{1 + (n-1)p_g}{\alpha_g + \alpha_s\,(n-1)\,g^2 p_g} \quad \ldots\ldots (12)$$

or

$$B = \frac{Q\,[0.225\,f_c' + f_s p_g]}{F_b\,[\alpha_g + \alpha_s (n-1) g^2 p_g]} \quad \ldots\ldots (13)$$

Note that a wide range of values is tabulated for both the numerator and the denominator for this last expression in Tables 3 and 4.

Discussion of this paper should reach ACI headquarters in triplicate by Nov. 1, 1958, for publication in the March 1959 JOURNAL.

CONCRETE BRIEFS

Notes from Field and Office

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Resistance of Portland Blast Furnace Slag Cement Concrete to Ice Removal Action

By W. C. HANSEN*, R. P. VELLINES†, and W. W. BRANDVOLD‡

INTRODUCTION

Portland blast furnace slag cement (Type IS and Type IS-A) has been in use in the United States for several years. It seemed desirable to compare in field tests the resistance of concrete made with Type IA and Type IS-A cements to freezing and thawing accompanied by the removal of ice with chemicals. Accordingly, during the 1956-57 winter, an investigation was undertaken similar to that reported by Hansen§ in 1954.

CEMENTS AND AGGREGATES

The six cements used in this study were the Type IA and Type IS-A cements from Plants A, B, and C. These were identified as A IA, A IS-A, etc. The compositions were normal for such cements. The variations of the principal components for the Type IA cements were: CaO, 62.1 to 63.6 percent; SiO_2, 20.8 to 21.6 percent; and Al_2O_3, 5.7 to 6.4 percent. For the Type IS-A cements, they were: CaO, 53.9 to 55.0 percent; SiO_2, 24.8 to 27.6 percent; and Al_2O_3, 7.5 to 8.8 percent. The results of the usual ASTM specification tests on the cements are given in Table 1.

The aggregates were sand and gravel from Elgin, Ill., which were stockpiled in a saturated surface wet condition.

SPECIMENS

There were four sets of specimens, each consisting of one 36 x 36 x 6-in. slab, one 6 x 6 x 32-in. beam, and five 6 x 12-in. cylinders. Fig. 1, 2, 3, and 4 show two of the slabs made with Type IA and two of the slabs made with Type IS-A cements after 20 cycles of ice removal. The slabs were built with a dike on each edge, to permit impounding of water on the slab. As soon as the finish brooming was completed, 1 x 1-in. dikes of 1:2 mortar made with Type IA cement were placed around the edges of the slabs. These were anchored to the slabs by small brass screws with the threaded end embedded in the slab and the head embedded in the dike. The slabs were cured 1 day under wet cotton mats and under wet sand for 6 days.

The control cylinders and the beam were stored beside the slab under wet burlap for 24 hr, after which the cylinders were removed from the molds and placed in the fog room

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†Member American Concrete Institute, Manager Research Laboratories, Universal Atlas Cement Co., Buffington, Ind.
‡Section Leader, Research Laboratories, Universal Atlas Cement Co., Buffington, Ind.

§Hansen, W. C., "Effect of Age of Concrete on Its Resistance to Scaling Caused by Using Calcium Chloride for Ice Removal," ACI JOURNAL, Jan. 1954, *Proc.* V. 50, pp. 341-352.

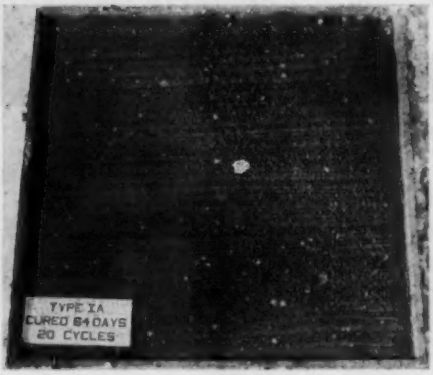


Fig. 1—Type IA cement concrete cured 64 days, shown after 20 cycles of ice removal

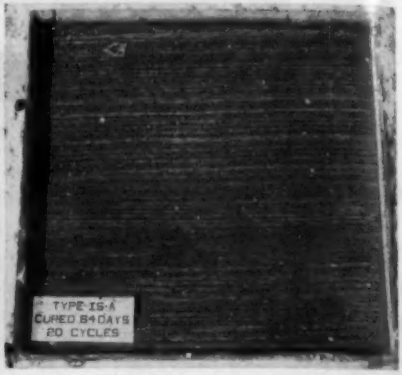


Fig. 2—Type IS-A cement concrete cured 64 days, shown after 20 cycles of ice removal

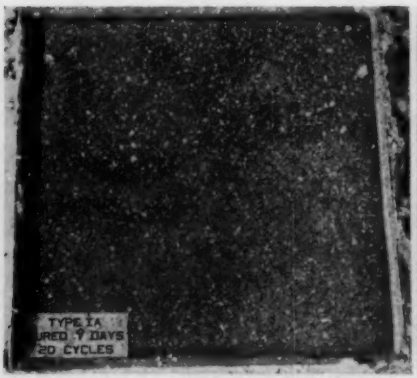


Fig. 3—Type IA cement concrete cured 9 days, shown after 20 cycles of ice removal

Fig. 4—Type IS-A cement concrete cured 9 days, shown after 20 cycles of ice removal

until tested. The beam was removed from the mold at 24 hr and embedded in the soil near the slab with the top surface as cast flush with the top of the soil. It was cured under wet sand 6 days. At the time of the first hard freeze, the beam was transferred to the laboratory fog room for at least 2 days, and was then broken in flexure, using third-point loading on an 18-in. span.

FREEZING AND THAWING

Specimens for each cement were made on September 13, September 27, October 11, and November 13 so as to be able to study the effect of time of placing on the resistance of the concrete to the removal of ice with salt. It has been reported that some liquids, such as ethylene glycol, that are used in deicing airplane wings behave like salts in causing scaling of concrete under freezing conditions. After the slabs had been made, it was decided to include ethylene glycol as a deicing agent, along with calcium chloride. Accordingly, the two slabs made with the cements from Plant A on September 13th were deiced with ethylene glycol and those made with the other four cements were deiced with calcium chloride. The slabs made on September 27th with cements from Plant

B and those made October 11th with cements from Plant C were deiced with ethylene glycol.

Water was placed to a depth of approximately ¼ in. on each slab shortly before the first freeze was expected. Whenever the water was frozen solidly, it was removed in the morning by the application of the equivalent of 2 lb per sq yd (1.85 lb per slab) of calcium chloride or 1/3 gal. of ethylene glycol per slab. These quantities of calcium chloride and ethylene glycol with the quantities of ice on the slabs would melt the ice at temperatures as low as about 5 F. After a few hours, the solution was flushed from the slab and replaced with fresh water.

At the end of each five cycles of freezing and thawing, the degree of scaling of the slabs was determined. This was done by placing a wire grid, divided into 25 squares, over the surface of the slab and estimating the scaled area in each of these squares. One square equaled 4 percent of the total. Working with these small areas permitted rather accurate estimates of the amount of scaling.

The gravel aggregate used in this study contained some pieces of chert. Usually, the first scaling on these slabs consisted of pop-outs over particles of chert or other pieces of gravel which were very close to the surface. This may be seen in Fig. 1 and 2.

Since these slabs were covered with water prior to freezing and continuously thereafter during the period of the experiment, the concrete must have been nearly saturated with water very soon after the first freezing. In the earlier experiment, Hansen pointed out that concrete pavements will rarely, if ever, be saturated for periods of several weeks in service nor are they likely to receive 25 cycles of freezing and thawing in a saturated condition in one winter. Accordingly, this experiment was continued for 35 cycles of ice removal so as to make the conditions much more severe than could ever be expected in service.

RESULTS

Scaling at 35 cycles did not exceed 5 percent on any of the slabs placed in September and October and occurred in isolated patches over particles of aggregate, as shown in Fig. 1 and 2.

All slabs placed November 13 were scaled to the extent of at least 40 percent by the end of 15 cycles. The first hard freeze occurred November 23. The condition of two of these slabs at the end of 20 cycles of ice removal is shown in Fig. 3 and 4.

The ages in days at the first freeze for the four sets of slabs were 64, 50, 36, and 9, respectively.

The average flexural strengths of beams cured with the slabs in the field at ages of 67, 53, 39 and 15 days were, respectively, 600, 580, 535, and 470 psi for the Type IA cement

TABLE 1—RESULTS OF PHYSICAL TESTS ON CEMENTS AND MORTARS

Properties tested	Cement					
Cement properties	A IA	A IS-A	B IA	B IS-A	C IA	C IS-A
Normal consistency	24.8	27.6	26.2	26.0	24.6	27.0
Setting time, hr and min						
Gillmore—Initial	2:40	3:40	3:00	3:15	4:05	3:20
—Final	6:35	7:00	6:00	6:45	7:10	6:50
Vicat set	1:40	2:35	1:45	2:00	3:10	2:10
Passing 325 mesh, percent	91.3	97.2	93.0	96.9	92.0	98.2
Specific surface, sq cm per g	1923	2565	1955	2510	1870	2495
Air content, percent	21.1	19.6	20.2	19.3	17.6	18.1
Properties of ASTM C 109 mortars						
Water, percent	43.9	44.6	44.6	44.3	44.2	46.6
Flow, percent	104	113.5	112	113	107	109.5
Compressive strength, psi*						
3 days	1765	2117	1893	1994	1725	2642
7 days	2917	3108	2825	3225	2967	3875
28 days	4467	5846	4250	5633	4825	6875

*Average of three 2-in. cubes.

concrete and 615, 545, 575, and 430 psi for the Type IS-A cement concrete.

The results of this study did not show any differences from the standpoint of durability of the concrete between the cements from the different plants nor between the Type IA and Type IS-A cements. Also, no difference could be detected between the action of calcium chloride and ethylene glycol.

ACKNOWLEDGMENT

The authors take pleasure in acknowledging the assistance of their colleagues who carried out the experimental work of this project.

Tabulation for Bar Selection

By PAUL ROGERS*

Reinforced concrete design handbooks contain tables of areas and perimeters of bars (A_s, Σ_o) for selection of reinforcing.

An additional tabulation, giving the ratio of Σ_o/A_s was found very helpful by the writer in the immediate selection of the proper size bars which satisfy the optimum requirements. For A_s and Σ_o, thus

Bar size	Σ_o	A_s	$\frac{\Sigma_o}{A_s}$
# 2	0.786	0.05	15.70
# 3	1.178	0.11	10.70
# 4	1.571	0.20	7.86
# 5	1.963	0.31	6.33
# 6	2.356	0.44	5.36
# 7	2.749	0.60	4.58
# 8	3.142	0.79	4.00
# 9	3.544	1.00	3.54
#10	3.990	1.27	3.14
#11	4.430	1.56	2.84

Example 1

Column footing: $N = 300\ K$; column size, 16 in. square; footing 8 x 8 ft, 22 in. thick; $d = 18$ in; $f_c' = 2500$; $f_s = 1125$; $v = 75$ psi; $u = 200$ psi; $f_s = 20{,}000$ psi:

$$p_{max} = \frac{317.5}{8^2} = 4.96 \text{ kips per sq ft}$$

$$p_{stress} = \frac{300}{8^2} = 4.68 \text{ kips per sq ft}$$

$$V_{shear} = 6.17 \times 1.83 \times 4.68 = 52.8 \text{ kips}$$

$$v = \frac{52{,}800}{12 \times 4.34 \times 0.866 \times 18} = 65 \text{ psi}$$

$$V_{bond} = 3.33 \times 8 \times 4.68 = 124.7 \text{ kips}$$

85 percent = 106 kips

$$\sum_o = \frac{106}{0.200 \times 0.866 \times 18} = 34 \text{ in.}$$

$$M = V_{bond} \times \frac{3.33}{2} = 124.7 \times 1.67 = 206 \text{ ft-kips}$$

85 percent = 177 ft-kips

$$d \text{ (required)} = \sqrt{\frac{206}{8 \times 0.196}} = 11.5 \text{ in.}$$

d (furnished) = 18 in.

$$A_s = \frac{177}{1.44 \times 18} = 6.83 \text{ sq in.}$$

$$\frac{\sum_o}{A_s} = \frac{34}{6.83} = 4.98$$

Select #6 bars. Use 16 #6 bars (A_s = 7.04; $\sum_o$ = 37.7).

*Member American Concrete Institute, Consulting Engineer, Chicago, Ill.

Example 2

Continuous girder: $b' = 15$ in; $d = 25$ in.; $t = 28$ in.; $f_c' = 3000$ psi; $f_e = 1350$ psi; $f_s = 20{,}000$ psi; $u_{bot} = 300$ psi; $u_{top} = 210$ psi; $M_{pos} = 150$ ft-kips; $M_{neg} = 182$ ft-kips; $V = 72$ kips.

$$A_{s_{pos}} = \frac{150}{1.44 \times 25} = 4.16 \text{ sq in.}$$

$$\sum o_{pos} = \frac{72}{0.30 \times 0.87 \times 25} = 11.1 \text{ in.}$$

$$\frac{\sum o_{pos}}{A_{s_{pos}}} = \frac{11.1}{4.16} = 2.67$$

Use two #10 bars and one #11 bar.

$$A_{s_{neg}} = \frac{182}{1.44 \times 25} = 5.05 \text{ sq in.}$$

$$\sum o_{neg} = \frac{72}{0.21 \times 0.87 \times 25} = 15.8 \text{ in.}$$

$$\frac{\sum o_{neg}}{A_{s_{neg}}} = \frac{15.8}{5.05} = 3.13$$

Select #10 bars; use four #10 bars.

No Design Without Inspection

Almost daily we hear of poor construction performance because of inadequate inspection. Such failures occur despite good designs and the availability of ACI recommended practices and the *ACI Manual of Concrete Inspection*. Committee workers who prepare these guides for obtaining good results with concrete realize that their efforts are effective toward that end only as their recommendations are carried out and confirmed by forthright inspection. Such inspection must be interpreted in the broadest sense, embracing the full range of authority, from management policy through design and field engineering, to the shift inspector. ACI Committee 611, Inspection, appreciates permission to reprint the following editorial which appeared in the Nov. 21, 1957 issue of *Engineering News-Record*.

Field inspection should be a part of every design contract. That it is not, is a cause for concern since many troubles—and some failures—could have been avoided if engineers had been present to insure that their designs were not misinterpreted.

None of the reasons that have been advanced for not doing the field inspection on his own designs seems strong enough to justify an engineer's risking his reputation by following such a course. If an owner won't pay reasonable inspection costs or insists on inspection by someone else, even after the dangers of such actions are explained, then the design contract can be refused. If the designer doesn't want to be bothered with field inspection, he will be extremely fortunate if he isn't bothered by more unpleasant duties as a consequence. If his office work keeps him too busy, he should give first preference to his field-inspection obligations before taking on new design assignments. Finally, if an owner argues that contractors won't take a job that is under an engineer's supervision, that job, too, can be refused.

Experience has shown that it is folly to assume that engineering drawings are so perfect that the designer's wishes can be followed without guidance. Many a job has gone wrong because the designer was not there when the lines he put on paper were translated into a structure. Design without designer inspection of construction is a hazard that need not and should not exist.

Problems and Practices

A series relating to "down-to-earth, everyday" concrete problems which attempts to give brief answers to the more common (and sometimes uncommon, too) questions asked about concrete design and construction practices.

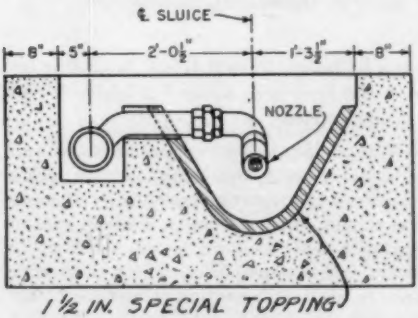
To some, the answers will seem simple and obvious; others may prove controversial.

All ACI members are invited to participate—either by submitting an inquiry, or even better, by telling JOURNAL readers how an intriguing problem was solved. It may well be that readers will be able to suggest more practical solutions than those presented.

Q. What kind of concrete is suitable for lining sluiceways carrying water-borne industrial wastes?

A. The sluiceway at the Rouge Plant Foundry, Ford Motor Co., Dearborn, Mich., (shown here in cross section) effectively withstands abrasion from waterborne cupola slag and dust, discharged at a constant driving pressure of 60 psi.

Several hundred lineal feet of these sluiceways are in operation 16 hr a day, 5 days a week. Designed by Ford engineers, the



sluiceways have given 6 years of trouble-free service. Water jets are spaced at 10-ft intervals.

Sluiceway surfaces are made by vigorously raking and grooving the structural base, soon after it is cast. Immediately following the raking, the vertical and horizontal areas are given a slush coat of portland cement and water. A low water ratio concrete mix of Cortland emery aggregate and portland cement is then applied and tamped in place to a thickness of 1½ in.

Maintenance has been confined to a few isolated points where the initial blasts from the jets occur.

* * *

Q. The use of control joints in concrete block walls to prevent shrinkage cracking is described in the paper "Control Joints Regulate Effects of Volume Change in Concrete Masonary," ACI JOURNAL, July 1957, Proc. V. 54, pp. 59-70. Are there any similar recommendations available for use of joint reinforcement instead of control joints?

A. The phrase "instead of control joints" dictates the answer which is no. The authors of the paper in question make definite recommendations for use of joint reinforcement *together* with control joints, but feel that it would be uneconomical to use sufficient reinforcement to eliminate the need of control joints, except in special design for earthquake, hurricane, or blast resistance. They do advise joint reinforcement for panels (1) containing intermediate weak spots like door or window openings, (2) under severe exposure such as parapets, or (3) designed for unusual lateral loads such as hurricane, blast, earthquake, or as heavily loaded retaining walls. This reinforcement is intended merely to hold the panel together between control joints. Spacing of control joints is not intended to be increased with use of ordinary amounts of such reinforcement. Note the long-standing recommendation of the Portland Cement Association for use of control joints even in monolithic concrete walls with minimum (¼ percent) temperature reinforcement, 50 percent of which is to be cut at the joints.

The only condition under which generally accepted practice would endorse omission of control joints in a concrete wall is under prestress, most common for circular tank walls.

CURRENT REVIEWS

of Significant Contributions in Foreign and Domestic Publications

Bridges

Bridge structures on the automobile highway Ljubljaba-Zagreb (in Slavonian)

S. Lapajne, *Gradbeni Vestnik* (Ljubljana), V. 8, No. 47-50, 1956-57, pp. 151-158

Reviewed by J. J. Polivka

Standard bridges are discussed, especially their economy compared with new bridge types in reinforced and prestressed concrete and with recently developed structural types, as e.g. flat slabs, long span slabs with two main girders without cross beams, etc.

On the design bending moment of highway beam bridge

M. Naruoka and H. Yoneyawa, *Proceedings*, 6th Japan National Congress for Applied Mechanics (Tokyo), 1956, pp. 165-169

Applied Mechanics Reviews Apr. 1958 (C. P. Siess)

Simplified approximate expressions for design moments in slab and beams of slab-and-beam highway bridges are based on analyses by theory of orthotropic plates. Different expressions are given for moments in edge and interior beam and for bridges with and without diaphragms or cross beams.

Construction of reinforced concrete balanced cantilever bridges over the Dhela River in U. P.

M. C. Johorey, *The Indian Concrete Journal* (Bombay), V. 32, No. 4, Apr. 1958, pp. 106-110

Describes the design, details, and construction of two new reinforced concrete cantilever bridges. Interesting features included prepacked concrete in the piers using colloidal grout and the use of reinforced concrete roller and rocker bearings instead of steel. Due to the excessive cost and slow delivery of such hardware in India, the bearings chosen were found to be cheaper and incidentally much lighter than the steel bearings generally used. The paper includes a complete description of these bearings, design calculations, and structural detail drawings.

Strength of right prestressed concrete slab bridges with edge beams

G. C. Reynolds, *Magazine of Concrete Research* (London), V. 9, No. 27, Nov. 1957, pp. 141-144

Author's Summary

Two small prestressed concrete slab bridges with edge beams have been tested to failure under single concentrated loads, and the results are in close agreement with those predicted by the yield line method of analysis.

Construction

Torsional instability and the use of the Cruciani centering in the construction of long arches (L'instabilité à la torsion et l'emploi du cintre Cruciani dans la construction des grands arcs)

Frido Cruciani, *Travaux* (Paris), V. 42, No. 280, Feb. 1958, pp. 143-148

Author's Summary

The Cruciani type of centering, consisting of two laminated timber chords with timber diagonal members, secured by a system of threaded steel tie-rods with bolts, can be advantageously used for the construction of long spans. In the present article the torsional stability of the centering is investigated with the aid of an approximate method. The author then describes its application to the

A part of copyrighted Journal of the American Concrete Institute, V. 30, No. 2, Aug. 1958, *Proceedings* V. 55. Address P.O. Box 4754, Redford Station, Detroit 19, Mich. Where the English title only is given in a review, the book or article reviewed is in English. If it is followed by a foreign title the work reviewed is in that language. In those cases where the foreign title cannot conveniently be set in type or is not available, the language of the original article is indicated in parentheses following the English title. Copies of articles or books reviewed are **not** available through ACI. Available addresses of publishers are listed in the June "Current Reviews" each year. In most cases ACI can furnish addresses of publications added later.

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centering for the Varèse bridge, with a theoretical span of 101 m, a rise of 29.2 m, and a width of 12.5 m.

Tunnel linings

OLIVER DAWSON, *Transactions*, Institution of Civil Engineers of Ireland (Dublin), V. 83, 1957, pp. 17-32

Discusses the advantages of cast-in-place concrete, bolted cast iron, or precast block tunnel linings. Ten references to British literature.

Economical use of construction machinery in building (Wirtschaftliche Verwendung von Baumaschinen im Hochbau)

GERHARD ACHTERBERG, *VDI Zeitschrift* (Düsseldorf), V. 99, No. 18, June 21, 1957, pp. 790-796

Reviewed by ARON L. MIRSKY

This paper concerns material elevators, skips, cranes, buggies, and similar equipment for the handling of materials, and concrete mixers.

Economical roof constructions of wood, steel and reinforced concrete for dwelling construction (Wirtschaftliche Dachkonstruktionen aus Holz, Stahl und Stahlbeton für den Wohnhausbau)

GERHARD BRAUN, *VDI Zeitschrift* (Düsseldorf), V. 99, No. 18, June 21, 1957, pp. 783-789

Reviewed by ARON L. MIRSKY

Well illustrated study of comparative costs of peaked roofs. Conclusion: the wood roof is still cheapest; cost of wages, rather than cost of materials, is currently the decisive factor in the case of precast concrete structural roofing elements, but these offer promise for the future.

The most desirable construction material for building (Die wichtigsten Baustoffe für den Hochbau)

WALTER ALBRECHT, KARL EGNER, and GUSTAV WEIL, *VDI Zeitschrift* (Düsseldorf), V. 99, No. 18, June 21, 1957, pp. 797-803

Reviewed by ARON L. MIRSKY

Stone, mortar, concrete; steel, reinforced concrete; wood and timber products; plastics —all are discussed and compared.

This and the two preceding papers form a good survey of European (German) construction today, and the trend of thought for the future. While direct, immediate applicability to American construction is problematical, there is material for thought.

Design and construction of the printing works at Debden—Discussion

The Structural Engineer (London), V. 35, No. 4, Apr. 1957, pp. 156-161

Reviewed by C. P. SIESS

Discussion of paper in *The Structural Engineer*, V. 34, No. 4, Apr. 1956 (see "Current Reviews" Feb. 1957, p. 812). Concerns tendon profile in prestressed arch, edge supports of shell roof, precasting techniques, contract administration, etc.

Deep space truss gives plant room for air equipment

Engineering News-Record, V. 160, No. 19, May 8, 1958, pp. 46-48

Column-free areas desired by Texas Instruments, Inc., for its two-story Dallas plant were obtained with a hyperbolic paraboloid concrete roof and a concrete space truss between first and second stories. The story-high truss—consisting of precast concrete diagonals, the second-floor slab, and the first-floor ceiling slab—provides a plenum with room for ducts and air handling equipment.

Precast concrete elevator

Concrete, V. 66, No. 3, Mar. 1958, pp. 32-33

Describes and pictures design and construction of a precast concrete silo 40 ft high with inside diameter 18½ ft. The walls were cast in six full-length elements, one upon the other, the first being cast upon shaped earth as a form. The flat roof, including upturned beams, was cast in three segments. Plastic sheets placed upon each slab were used as bond breakers. The units were joined together by bolts in steel clip angles and sealed with rubber strips full length.

French projects in South Africa

French Economic and Technical Bulletin, No. 11, 1957, pp. 14-16

Describes a standardized construction method for building thin shell reinforced concrete water-cooling towers. The structures are about 328 ft high, hyperbolic in profile, with diameters from 118 ft at the neck to 197 ft at the base.

Example of a large-space structure in reinforced concrete and its analysis (in Croatian-Serbian)

D. Zlokovic, *Naše Gradevinarstvo* (Belgrade), V. 12, No. 1, Jan. 1958, pp. 42-45

Reviewed by J. J. Polivka

Structure spanning 150 ft, 42 ft clearance, with long overhangs and balconies was designed by the author for the Belgrade Fair. In this connection, various structural types were investigated as to their illumination, economy and other conditions. The effect of butterfly shape frames, spaced 30 ft, and natural illumination through slightly curved roof surfaces and vertical glass walls is valuable contribution to the esthetics of the building. Structure was built with 3.3 cu ft concrete, and 4.6 lb steel per sq ft.

Water tower at Caen-LaGuériniére (Réservoir de La Guériniére

A. Darde, *Travaux* (Paris), V. 42, No. 280, Feb. 1958, pp. 125-130

Author's Summary

The water tower at Caen-La Guériniére presents a highly original solution of the esthetic treatment of water towers. The structure comprises, in addition to the tank of 3000 cu m capacity, a covered market and administrative offices. It consists of vertical reinforced concrete diaphragms intersecting on the axis of the structure. These carry concrete shells which form the walls of the tank and the roof of the covered market.

Construction Techniques

Concrete curing time reduced by means of electric current (in Polish)

T. Latomski, *Przeglad Budowlany* (Warsaw), No. 10, 1956, pp. 423-424

Polish Technical Abstracts No. 4 (28) 1957

Research over reducing concrete curing time by means of electric current has been carried out at the Building Organization and Mechanization Institute. The item tested was a flight of stairs having a volume of concrete amounting to 0.33 cu m. The external temperature during the heating process was −5 C, the maximum temperature of heating up to 70 C. A single 16-hr heating process was applied. The reinforcement constituted one electrode, surface electrodes comprising the other. The maximum voltage applied was 100 V. Forms were removed when the temperature of the concrete in the structure reached about 25 C. Total consumption of electric energy was 21 kwh, which corresponds to an average consumption of 63 kwh per cu m. Heating by means of electric current in this case proved cheaper than steam.

Quality control of concrete

J. M. Fisher, *International Civil Engineer and Contractor* (London), V. 10, No. 2, Mar.-Apr. 1958, pp. 44-51

Discusses quality control of concrete construction in pavements for roads and airfields. Particular emphasis is placed upon the problems of uniform compaction to secure uniform field density of the concrete and thereby to permit finishing to close tolerances for improved riding qualities in the pavement. The control of formwork and construction practices and equipment required for finishing and testing finish are discussed.

Resistance of concrete pumped through pipelines

A. N. Ede, *Magazine of Concrete Research* (London), V. 9, No. 27, Nov. 1957, pp. 129-140

Author's Summary

An investigation of the basic mechanics of concrete pumping. Unsaturated materials pass stresses by interparticle contact and give rise on commencement of movement to a pressure build-up which in a nontapering duct is exponential with distance pumped. Adequate saturation by water-cement paste reduces the resistance to a favorable linear type, subject to increase with velocity, but independent of operating pressure.

Test of holding strength of form anchors embedded in concrete placed and cast at reduced temperatures

Ralph A. Bendinelli, *Technical Report* No. 6-473, U. S. Army Engineer Waterways Experiment Station, Vicksburg, Miss., Feb. 1958, 13 pp., $0.50

Author's Summary

Tests were made to determine the shear resistance or holding strength of form anchors placed in concrete mixed and cured at reduced temperatures (45-50 F).

Four mass (6-in. maximum size aggregate) concrete blocks were cast and cured in a temperature-controlled room, and six 18-in. and sixteen 24-in. form anchors were em-

bedded in the blocks and tested at an age of 72 ± 4 hr. One of the blocks contained a 30 percent replacement (by solid volume) of the portland cement with fly ash.

The results indicate that adequately designed anchors of either length will, under the test conditions used, provide resistance at 3 days age in excess of normally expected loads even though the concrete has a cement content as low as 2.5 bags per cu yd and is placed and cured at reduced temperatures. The use of fly ash as a replacement material reduced the holding power of the anchors appreciably, but not below a safe level.

Dams

Seismic vibrations on the dams (Le axion sismiche sulle dighe)

FRANCESCO PEUTA and GUILIO SUPUIO, *Giornale del Genio Civile* (Rome), V. 95, No. 7, 8, July, Aug. 1957, pp. 499-517

Reviewed by JOHN I. PRELEE

The authors examine the effect of externally caused vibrations on the design of concrete dams, and the point at which the action factor should be considered as dangerous. They comment on the design of different dams; theory of Westergaard is also discussed.

Design

New theory for the design of thin shells carrying load in the longitudinal as well as the transverse direction (Théorie nouvelle de calcul des voutes autoportantes)

J. BARETS, *La Technique Modern-Construction* (Paris), V. 12, No. 8, Aug. 1957, pp. 236-239; No. 10, Oct. 1957, pp. 309-312

Reviewed by ALEXANDER M. TURITZIN

Purpose of the article is to present a simplified flexure theory of parabolic roof shells without resort to complex calculations used in the design of thin shells by the so-called membrane theory. The new theory leads to a redistribution of shear stresses and a reduction of transverse moments usually neglected in the analysis by the membrane theory. Transverse moments are computed for a single parabolic shell, for multiple shells, and for shells with vertical and horizontal stiffening girders placed along longitudinal edges. The author believes that a simplified theory is necessary at the present time, since the application of the membrane theory is too involved and time-consuming for a structural designer.

Determination of the deflection at midlength and the required moment of inertia of flexural members of arbitrary type (Bestimmung der Mittendurchbiegung und des erforderlichen Trägheitsmomentes bei Biegestäben beliebiger Tragwerke)

M. SCHINEIS, *Der Bauingenieur* (Berlin), V. 32, No. 5, May 1957, pp. 169-172

Reviewed by ARON L. MIRSKY

Usual (elementary) formulas for deflection can be written, after some manipulation, as functions of the bending moment, span, and two coefficients α and β:

$$I_{req'd} = \varphi \cdot \alpha \cdot Ml$$

$$= \beta \cdot Ml$$

$$\delta_{actual} = \alpha \cdot Ml^2/I$$

$$\frac{\delta}{l}_{actual} = \alpha \cdot Ml/I$$

where $\varphi = \dfrac{l}{\delta}$.

This interesting paper presents an extensive tabulation of bending moment diagrams and values of α and β (in metric units) for single spans with various types of loading and support conditions and of four materials (steel, aluminum, reinforced concrete, and timber). Examples of the use of these tabulated values in the analysis of more complicated structures are given: a continuous girder, a portion of a multistory building frame, a two-hinged rigid frame, and a single-span girder with variable moment of inertia.

Design of masonry walls for blast loading

K. E. MCKEE and E. SEVIN, *Proceedings*, ASCE, V. 84, ST 1, Jan. 1958, pp. 1512-1–1512-18

AUTHORS' SUMMARY

The arching action theory of masonry walls is applied to the problem of blast resistant design. Based on this theory, an equation of motion is developed for a masonry beam of solid cross section restrained by essentially rigid supports. This equation is solved for a simplified but realistic form of blast loading.

The results obtained permit either the design of a wall for a given loading or the determination of the maximum loading which a given wall can withstand. The theory is extended by an approximate method to wall panels supported on four sides. Comparison is made with test data for walls subjected to full scale atomic and high explosive blasts.

A method of design of reinforced concrete sections

PANAGIOTIS D. MOLIOTIS, *Proceedings*, ASCE, V. 84, ST 1, Jan. 1958, pp. 1509-1–1509-25
AUTHOR'S SUMMARY

A general solution is given of the calculation of any section of reinforced concrete with an axis of symmetry, eccentrically loaded, by the use of computation formulas of the rectangular section and coefficients defined in terms of the statical moment and moment of inertia of the compressed area of concrete.

Vibrating bodies with several degrees of freedom (Schwinger mit mehreren Freiheitsgraden)

EUGEN DOEINCK, *Der Bauingenieur* (Berlin), V. 32, No. 5, May 1957, pp. 165-169
Reviewed by ARON L. MIRSKY

Two topics in undamped harmonic vibrations are studied with the aid of the "equivalent static load": (1) the characteristic frequency in free vibration, and (2) the stresses in forced vibration.

Some experiments on the applicability of the principle of superposition to the strains of concrete subjected to changes of stress, with particular reference to prestressed concrete

R. D. DAVIES, *Magazine of Concrete Research* (London), V. 9, No. 27, Nov. 1957, pp. 161-172
AUTHOR'S SUMMARY

The paper describes experiments to test the validity of McHenry's hypothesis that the principle of superposition can be applied to the strains, including the creep strains, of concrete subjected to changes of stress. It is concluded that, although the hypothesis is by no means strictly correct, it is nearer the truth than the assumption that change of strain is proportional to change of stress. The application of the hypothesis to prestressed beams under sustained loads is considered with reference to the distribution of stress on a cross section and to bending.

Estimation of the critical loads of certain frameworks

H. G. ALLEN, *The Structural Engineer* (London), V. 35, No. 4, Apr. 1957, pp. 135-140
Reviewed by C. P. SIESS

Describes method of "successive substitution" in which members are removed one by one, starting at an extremity, and the properties of the remaining members are modified so that the joint stiffnesses in the remainder of the truss are unaltered. Process is repeated until only one modified member or group of members remains. If these are stable, the original framework is stable. For continuous beams, plane braced frameworks, Vierendeel girders, symmetrical single bay multistory building frames, and similar structures, this method is claimed to be easier to apply than other methods such as condensation of determinants or moment distribution.

Savings to be effected by the more rational design of cased stanchions as a result of recent full-size tests—Discussion

The Structural Engineer (London), V. 35, No. 4, Apr. 1957, pp. 148-155
Reviewed by C. P. SIESS

Discussion of paper by Oscar Faber in *The Structural Engineer*, V. 34, No. 3, Mar. 1956, pp. 88-109 (see "Current Reviews" Mar. 1957, p. 912). This paper, describing extensive tests and recommending a new British design formula for composite columns, received extensive discussion relating to practical problems of construction, load transfer from beams to columns, and to the appropriateness of the design formula proposed, especially for slender columns.

Effects of wall deformations on floor slab loads and floor slab deformations in multistory houses (in Swedish)

HENRIK NYLANDER and EINAR ERIKSON, *Nordisk Betong* (Stockholm), V. 2, No. 1, 1958, pp. 91-110
Reviewed by MARGARET CORBIN

Load distribution is analyzed for three-story and six-story structures. Tests were conducted at the Royal Institute of Technology, Stockholm, on lightweight walls subjected to short-time loads and on brick masonry wall piers, subjected to long-time loads. The portion of the total ground floor load transmitted by non-underpinned light-

weight walls varied from 2 to 11 percent. When the lightweight walls were underpinned, they transmitted 15 to 30 percent of the total load. The risk of crack formation in lightweight walls becomes much greater if the direct underpinning of these walls by the floor slab decreases or is partly neutralized, i.e., owing to large deformations of the slab. In unfavorable cases the force acting on the lightweight walls caused the load on the lowermost floor slab to become twice as great in the three-story buildings and three times as great in the six-story buildings.

Influence of the tension ring on the analysis of water tanks and sewage towers (Der Einfluss des Zugringes auf die Berechnung von Wasserbehältern und Faultürmen)

JOACHIM BORN, *Der Bauingenieur* (Berlin), V. 32, No. 5, May 1957, pp. 157-161

Reviewed by ARON L. MIRSKY

Several numerical examples involving a cylindrical tank with a conical shell roof are used to show that a tension ring (which may be shaped to double as a transition zone from cone to cylinder) has a considerable and desirable influence stresswise. Methods of handling prestressing, or of including effect of flexural stiffness of the ring, are also indicated. (For a correction to Table I of this paper, see issue of July 1957, p. 284.)

New application of the polygonal vaults theory in the computation of arbitrary cylindrical shells (in Polish)

W. ZALEWSKI and J. KRZEMINSKI, *Inzynieria i Budownictwo* (Warsaw), No. 3, 1957, pp. 137-146

POLISH TECHNICAL ABSTRACTS No. 3 (27) 1957

A polygonal vault inscribed into a curve of the transverse cross section is computed by two simplified methods which take into account the rigidity of boundary elements. The first method consists of a system of equations of the continuity of the transverse cross section with unknown values of the edge moments (the starting point here is the equation of three normal stresses acting along three neighboring edges). The number of equations corresponds to the number of edges. With the second method, the unknown values in the system of equations represent the displacements of the individual polygonal vault in their plane. The ultimate result is obtained by a system of linear equations representing the relation between total forces causing actual unknown displacements and external loading forces.

Statics of reinforced concrete shells of cooling towers (in Polish)

J. LEDWON, *Inzynieria i Budownictwo* (Warsaw), No. 3, 1957, pp. 101-107

POLISH TECHNICAL ABSTRACTS No. 4 (28) 1957

Shells used as exhaust ducts of cooling towers fulfill conditions enabling the adoption of the momentless shell theory. The effect of a support or rigidity of the shell is regarded as disturbance of the boundary conditions. Hyperboloidal shells are computed for the individual horizontal cross sections. Thus are avoided difficulties in the computation, arising from a continual change in the shell thickness. The computation of internal forces, due to dead load, is based for all types of shells on the application of common equations of equilibrium. Thermal effects are taken into consideration only as irregular heating of the wall, with a difference in the temperature of the medium amounting to 50 C.

Pin-ended gabled frames

J. CHINN, *Proceedings*, ASCE, V. 83, ST 5, Paper 1353, Sept. 1957, 13 pp.

APPLIED MECHANICS REVIEWS Apr. 1958 (P. Csonka)

Paper presents a method suitable for the calculation of one- or two-span pin-ended gabled frames. The new method applies coefficients known from the moment-distribution method and formulas used in the slope-deflection method. In case of one-span frames, the solution of the problem claims for the solving of one, or in case of two-span frames, of three simultaneous equations.

Industrial floor slabs: design and construction

J. L. STAUNTON, *Architectural Record*, V. 123, No. 5, May 1958, pp. 245-249, 326, 330, and 334

A survey in the January *Architectural Record* indicated that difficulties with industrial slabs on grade are widespread. It also indicated types of failures which occur, and prevailing practices in design and construction. This paper discusses some of the various slab problems and presents a step-by-step procedure

for more satisfactory design and construction. A rational design method is recommended in designing for industrial truck loads. Design formula and construction methods recommended by ACI Committee 325 and the Portland Cement Association are suggested.

Calculus regarding the breaking point of a bridge structure with rigid arch (Il tema di calcolo a rottura. Risultati sperimentali relati ir ad una struttura da poute a volto irrigidita.)

VITTORIO SORGENTE, *Giornale del Genio Civile* (Rome), V. 95, No. 12, Dec. 1957, pp. 1099-1109
Reviewed by JOHN I. PRELEZ

Testing the failure point of a concrete slab bridge supported by a concrete shell type arch. The author presents with clear diagrams, calculations, and pictures the different reactions and moments occurring in the top concrete slab and the lower concrete arch structure under different weights until reaching the collapsing point.

Design of catenary and paraboloidal membrane roofs

S. SIVASWAMI, *The Indian Concrete Journal* (Bombay), V. 32, No. 2, Feb. 1958, pp. 41-45 and p. 68

Derives a method of analysis and shows design example for a low cost individual shell house. The construction method is described and a brief specification supplied. The shell consists of an inner shell of plain concrete 2 in. thick with a 3-in. double air gap and an outer shell of 1½ in. of cinder concrete. The outer concrete layer is waterproofed and apparently no conventional roofing is required.

Calculation of a wall-type girder according to the composite method (in German)

W. VALENTIN, *Beton-und Stahlbetonbau*, (Berlin) V. 52, No. 8, Aug. 1957, pp. 176-182
APPLIED MECHANICS REVIEWS May 1958 (Foppl)

For the sake of calculating a wall-type girder, the wall is cut in parallel strips, which are computed by the simple-beam theory. The beams must pass without open space and without a spring of the stresses in the intersections. These conditions require loads for the beams and shearing stresses in the intersections. The method is illustrated by some examples.

Vibration of cylindrical shells

K. HEKI, *Journal of the Institute of Polytechnics* (Osaka), Series F, Architecture; V. 1, No. 1, Nov. 1957, pp. 1-36

Presents an approximate solution for the vibration of cylindrical shells. Develops the theory, compares the results to more exact theories, and concludes that the approximate theory presented is sufficiently accurate for practical purposes and more appropriate due to its simplicity.

Precise method of moment distribution

A. E. HOLDAWAY, *Concrete and Constructional Engineering* (London), V. 53, No. 2, Feb. 1958, pp. 73-78

Presents a single cycle method of moment distribution. The method differs only slightly and lacks the clarity of the method which appeared in the paper "A Precise Moment Distribution Method" by J. A. Wise in the Nov., 1938 ACI JOURNAL ,(*Proc.* V. 35 pp. 93-112).

Materials

Heavy media separation boosts gravel value

I. S. THYLE and C. E. GOLSON, *Rock Products*, V. 61, No. 5, May 1958, pp. 126, 130, 132, 167, 168

Describes the heavy media separation processes now in use with cost figures for operation. In particular brings out economies possible in the concrete mix when using the higher quality aggregate.

Methods for rating concrete waterproofing materials

F. KOCATASKIN and E. G. SWENSON, *ASTM Bulletin*, No. 229, Apr. 1958, pp. 67-72

In an attempt to develop reliable test methods for evaluating the effectiveness of concrete waterproofers, it was found that a combination of saturated and unsaturated permeability tests is necessary, the latter covering more than one humidity condition.

The materials tested showed to no advantage in the saturated permeability test. However, this test may have some merit where filler-type or pozzolanic admixtures are used. The unsaturated permeability test appeared to make clear distinction between certain types of waterproofings and could be related to certain field situations. The capillary absorption test gave a good indication of the

over-all effectiveness of waterproofings in cases where concrete is subjected to short periods of contact with moisture, with intermediate drying intervals.

The water repellent and bituminous admixtures showed up favorably in all the tests except the saturated permeability test. The calcium chloride admixture showed no beneficial effects in this study, apparently because the cement used did not respond to the usual accelerating action of calcium chloride on hydration.

The cement-base paints provided varying degrees of benefit when painted on the side of the specimen subjected to wetting. The silicone application had a significant influence on moisture flow.

New plant for calcium aluminate cement

Cement and Lime Manufacture (London), V. 31, No. 2, 1958, p. 29

Reviewed by R. P. VELLINES

Thumbnail description is given of plant erected at West Thurrock, Essex, England, for manufacture of a new white calcium-aluminate cement with an alumina content over 70 percent and iron compounds and silica impurities less than 1 percent. In addition to its refractory properties, the new cement will have same rapid hardening and corrosion resistance properties as high alumina cement. Oil-fired rotary kiln is lined with 7 in. of refractory concrete in its cooler sections; in the hot zone, 2 in. of insulating aggregate was placed against the steel shell and 5 in. of pure fused alumina was cast integrally with it.

Concrete aggregate examination by prolonged copper-nitrate staining test

L. DOLAR-MANTUANI, *Bulletin of the Geological Society of America*, V. 68, No. 12, Part 2, p. 1717, Dec. 1957

HIGHWAY RESEARCH ABSTRACTS May 1958

Applied over night the copper-nitrate staining test commonly used for distinction between calcite and dolomite not only provides means for identification of carbonate rocks or carbonate constituents and secondary carbonate products in silicate rocks, but helps to make the impurities in carbonate rocks, especially clayey material, much more visible. In addition, in argillaceous rock types suspected of being harmful in concrete, scaling, splitting and/or cracking are developed because of expansion. The effect of the test in this respect is analogous to the sodium or magnesium sulfate soundness tests widely used in the appraisal of concrete aggregates, but the former test requires a much shorter time to reveal some of the weak properties in certain rocks.

Attack on refractories in the rotary cement kiln

Reprinted from *Transactions of the British Ceramic Society*, V. 56, No. 5, May 1957, pp. 237-258, by the Commonwealth Scientific and Industrial Research Organization, Melbourne

Part 1—Physical and mineralogical changes

S. M. BRISBANE and E. R. SEGNIT, pp. 237-252

Part 2—The section C_2S-M_2S-C_3A_3 of the system CaO-MgO-Al_2O_3-SiO_2

E. R. SEGNIT and J. H. WEYMOUTH, pp. 253-258

The mineralogical changes occurring in magnesite-chrome and high-alumina refractories in the clinkering zone and in a siliceous firebrick in the calcining zone of a rotary cement kiln are studied. Some optimum conditions for the formation of a clinker coating are inferred. Large increases in the coefficient of thermal expansion were noted, which in the case of magnesite-chrome refractories appear to be associated with cracking.

Concretes containing blended reground portland cement (in Russian)

A. V. VOLZHENSKY and L. N. POPOV, *Beton i Zhelezobeton* (Moscow), No. 3, Mar. 1958, pp. 88-93

Reviewed by D. WATSTEIN

Experiments were performed with reground portland cements in the interest of reducing the coarser fraction of the cement (over 30 microns) which is said to account for 35 to 40 percent of the total weight of cement. It was observed that when such abrasive substances as quartz sand, slag, granite, and limestone are added to portland cement in amounts of 30 percent or more and the blend is reground in ball mills, the resulting binder is equal to or superior to the original portland cements. The comparative strengths were obtained for both mortars and concretes moist cured for 28 days.

Particularly favorable results were obtained for blends containing quartz sand (with impurities of less than 10 percent) and blast furnace slags. It was estimated that for water-cement ratio of 0.5 to 0.65, 1 ton of the

original portland cement yielded 1.35 to 1.5 tons of the reground blended cement of equal cementitious value in concretes. For concretes having a lower water-cement ratio, the "equivalent-cement" value increased to 1.5 to 1.8 tons for each ton of original cement.

The article contains tabulated data for blends ranging from 0.70:0.30 (portland cement admixture) to 0.15:0.85. Results on both moist cured and autoclaved concretes are reported.

Ten years of progress in gravel beneficiation—1948 to 1958

WILLIAM L. PRICE, *Circular* No. 71, National Sand and Gravel Association, Mar. 1958, 19 pp.

Four different beneficiation processes—heavy media separation, jigging, rising current classification, and elastic fractionation—have been introduced for the removal of soft, unsound, and otherwise objectionable particles of gravel. The development of each process in the sand and gravel industry is described in detail.

Properties of cement containing larger amounts of alumina and ferric oxide (in Japanese)

TOKUJI YAMANE, *Semento Gijutsu Nenpo*, No. 3, 1949, pp. 96-103

CERAMIC ABSTRACTS
May 1958 (Y. Suzukawa)

Cements containing 6.4 to 6.6 percent Al_2O_3 and 5.0 to 5.3 percent Fe_2O_3 were prepared. Their physical properties are compared with those of ordinary portland cement.

Considerations relative to standards and qualifications of portland cement based on standard tests (in Spanish)

JANOS MAGASREVY, *Revista del Colegio de Ingenieros de Venezuela* (Caracas), No. 261, Dec. 1957, pp 4-12
Reviewed by JOSEPH J. WADDELL

The author discusses the chemical compositions of cement, pointing out that cements with identical chemical composition may have different properties. Owing to these differences, the standards generally do not define the mineralogical composition of cement, instead they are limited to the content of minor components such as SO_3 and MgO.

The importance of gypsum is discussed. Tests are described which demonstrate that small variations in the gypsum content substantially change the mortar strength depending on the mineralogical composition of the clinker. Fineness is important, but the finest ground cement is not the best in all cases.

Present standards of the different specifications concerning the control of hardening are divided into two groups, those with damp mortar made with uniform sand, and plastic mortar made with graded sand. These tests are discussed.

The importance of contraction, especially of high quality concrete, is mentioned, as well as existing methods for its determination. Methods of determining heat of hydration and chemical resistance are discussed.

Methods of testing mortars for plastering and masonry adopted by the Scandinavian Mortar Committee (in Swedish)

Nordisk Betong (Stockholm), V. 2, No. 1, 1958, pp. 111-119
Reviewed by MARGARET CORBIN

The Scandinavian Mortar Committee's testing methods for masonry and rendering mortars are presented. These methods are intended for the testing of mortars sampled at the building site or prepared at the laboratory. Methods are given for the determination of the ultimate strength in flexure and compression, bond strength, shrinkage, capillary suction, water permeability, and soundness.

Method of evaluating the activity of pozzolans (in Japanese)

TOSHIYOSHI YAMAUCHI, *Semento Gijutsu Nenpo*, No. 2, 1958, pp. 83-89

CERAMIC ABSTRACTS
May 1958 (Y. Suzukawa)

Tests were made on five natural pozzolans found in Japan. Chemical analysis of the soluble portion, particle size distribution measurement, absorption test of CaO, and thermal analysis of the reaction products of the system pozzolan-$Ca(OH)_2$-H_2O are considered suitable methods for evaluating the activity.

Effects of fineness on the hydration of portland cement (in Japanese)

KIMBE CHUJO, *Semento Gijutsu Nenpo*, No. 1, 1947, pp. 87-100

CERAMIC ABSTRACTS
May 1958 (Y. Suzukawa)

Two kinds of ordinary cement clinker, (1) C_3S 48.6 percent, C_2S 31.4 percent and

(2) C_3S 60.5 percent, C_2S 21.1 percent, were ground to various finenesses having different particle size distributions with the addition of 3 percent by weight of natural gypsum, resulting in six samples. The heat of hydration and the amount of the combined water are correlated with the fineness, i.e., the degree of hydration. The shrinkage is due partly to the degree of hydration but principally to the evaporation of the semicombined water in the gels produced during hydration.

False set of cement (in Japanese)

KEIICHI MURAKAMI, SHUICHI IZUKA and HIROBUMI TANAKA, *Sekko to Sekkai* (Tokyo), No. 26, 1957, pp. 8-15

CERAMIC ABSTRACTS
May 1958 (Y. Suzukawa)

The effects of the form and amount of gypsum added on false set were studied by use of an improved Vicat needle. The degree of dehydration of gypsum was determined by differential thermal analysis. Differential thermal analysis of the mixture of clinker and gypsum showed that the transformation to hemihydrate occurs at 95 to 100 C, and that from hemihydrate to soluble anhydrite showed that the soluble anhydrite changes to hemihydrate after 10 to 15 hr exposure. This phenomenon seems to be a cause of false set of slightly aged cement.

Pavements

Scaling of concrete pavements in airfields of the Swedish Air Force Administration (in Swedish)

SVEN G. BERGSTROM and BJORN ORBOM, *Utredningar* (Applied Studies) No. 1, Swedish Cement and Concrete Research Institute (Stockholm), 1956, 34 pp.
Reviewed by MARGARET CORBIN

Results of a survey of existing pavements conducted in Sweden in 1953-1954 show that frequency of scaling of concrete pavements exposed to climatic influences alone and not subjected to salt treatment, is determined by circumstances as follows: (1) the combined effect of the number of winters, consistency, and dispersion in quality (strength) of concrete can be expressed by an equation presented by the authors; (2) the contractor's influence is decisive since he controls consistency and uniform quality; (3) if the workmanship is poor, the extent of deterioration depends also on the grade of cement; (4) air-entraining agents significantly improve frost resistance of pavements. No appreciable effect was observed which could be attributed to season of placing, cement content (within the limits of 300 to 350 kg per cu m), water-cement ratio, and strength. Scaling of slabs was observed in 7.9 percent of a pavement which received no salt application, and in 63.3 percent of another which received such application.

Preliminary experiment to compare the behavior of concrete beams under static and dynamic loading conditions

Research Note, RN/3116/JJT.ENF, Road Research Laboratory, Department of Scientific and Industrial Research, Harmondsworth, Middlesex, England
SUMMARIES OF ROAD RESEARCH NOTES
No. 57, Jan. 1958

Concrete road slabs are usually subjected to combinations of strains applied at various speeds. There are the *static* strains due to restraint of the warping caused by the temperature gradients which usually vary cyclically once per day, those due to thermal expansion and contraction which depend on the mean temperature of the slab and will vary daily and also seasonally, and *near-static* strains which result from shrinkage, changes of moisture content, etc. Superimposed on these slowly varying strains are the dynamic strains caused by vehicles moving across the slab.

The dynamic tests gave flexural strengths about 36 percent above the static value, while the strain at fracture was raised by about 20 percent. The nearly linear part of the load-strain curve, up to the onset of permanent deformation, was much longer in the dynamic tests than in the static ones, the corresponding limiting value of strain being twice that given by the static test.

Proposed experimental prestressed concrete road

Research Note, RN/3028/JPS, Road Research Laboratory, Department of Scientific and Industrial Research, Harmondsworth, Middlesex, England
SUMMARIES OF ROAD RESEARCH NOTES
No. 57, Jan. 1958

The outstanding requirement is a direct comparison of the performance on a heavily traveled road of prestressed with that of normal reinforced concrete. Before such an experiment is made, however, it will be necessary to obtain experience of the techniques of constructing a prestressed concrete

road and to assess the practical value of a bituminous sliding layer.

This note gives details of a proposal to lay two slabs in the laboratory's grounds at Langley Hall, each 120 ft long, 20 ft wide, and 6 in. thick and post-tensioned. One slab will be laid directly on the prepared foundation and the other on a sliding layer consisting of ¼ in. of 100-penetration bitumen.

Ten-year report on Illinois experimental continuously reinforced pavement

Roads and Streets, V. 101, No. 3, Mar. 1958, pp. 116, 117, and 121

Summarizes the report presented at the Highway Research Board annual meeting by J. D. Lindsay, engineer of materials, Illinois Division of Highways. Conclusions of the Illinois experience are that continuously reinforced pavements are economically sound on the basis of first cost, maintenance cost, service life, and salvage value and that steel percentages between 0.5 and 0.7 percent are adequate with a minimum of an 8 in. thickness.

Dutch motor road

The Engineer (London), V. 204, No. 5307, Oct. 11, 1957, pp. 540-541

Reviewed by Aron L. Mirsky

Description of Route 4A in the Netherlands, designed to link existing sections of superhighway between Amsterdam and the Hague and thus provide both a high speed motor route between them and a bypass for through traffic around the latter. Part of project is a major intersection with Route 12, which is projected for the future when traffic warrants.

Precast Concrete

Concrete railroad ties (in Danish)

S. Thornings Christensen, *Ingeniøren*, B Edition (Copenhagen), V. 67, No. 7, Apr. 1, 1958, pp. 229-235

Reviewed by Jesper Strandgaard

Describes history and development of concrete railroad ties in Europe, from Monier's first design in 1884, to the present types, among which the French, with two steel-bar-connected pads, Type RS, and the German, conventionally shaped, Type B 58, seem prevailing. The use of concrete ties is rapidly growing in Europe, and it appears, that the concrete tie now has left the experimental state and is here to stay.

Precast portal frames

C. F. C. Crisp, *Constructional Review* (Sydney), V. 31, No. 4, Apr. 1958, pp. 18-24

Describes design and presents various practical details for precast reinforced concrete rigid frames, with emphasis on the techniques of jointing.

Prestressed Concrete

Prestressed railroad ties, parts I and II

Concrete Products, V. 61, No. 4, Apr. 1958, pp. 38-43 and 72-80; No. 5, May 1958, pp. 20-25, 36, 38, and 44

Part I describes the development by various European countries of different types of precast and precast prestressed concrete railroad ties.

Part II discusses and describes advantages and disadvantages both in use and production of the types of units manufactured in France, Belgium, Germany, and England. Includes description of research program by American Association of Railroads on feasibility of prestressed ties for United States economy.

Twelve cu m prestressed, prefabricated tank (in Hungarian)

B. Gnadig, J. Thoma, *Magyar Epitöipar*, V. 6, No. 1-2, 1957, pp. 1-7

Hungarian Technical Abstracts V. 10, No. 1, 1958

The design and construction of a prefabricated, post-tensioned 1200 cu m tank of reinforced concrete is described. The 8.50 m high, 18.60 m diameter tank is covered by a shell dome. The sides are composed of 8 cm thick elements, weighing 2.3 tons each while the trapezoidal and triangular elements constituting the dome are 6 cm thick and weigh 0.7 ton each.

Fatigue in prestressed concrete structures

R. E. Rowe, *Constructional Review* (Sydney), V. 31, No. 3, Mar. 1958, pp. 18-22

Author's Summary

A review of the work on fatigue in prestressed concrete and observations by the various investigators.

Tests show the excellent fatigue resistance of prestressed concrete in the design range of

loading. It is apparent that for loadings outside the design range, i.e. after cracking has occurred, the governing factor affecting the fatigue strength of prestressed concrete is the fatigue strength of the high strength steel used for stressing. Figures obtained for the fatigue strength of prestressing wires show that for most types in present use a mean value of 60-65 percent of the ultimate strength should be taken. No comparable figures for the fatigue strength of high strength alloy bars are available.

Reinforced concrete headgears in South Africa

The Engineer (London), V. 204, No. 5312, Nov. 15, 1957, pp. 723-724

Reviewed by Aron L. Mirsky

Brief description of several mine headframes of reinforced and prestressed concrete. [*Cf* S. Macerata in ACI Journal, Sept. 1952, *Proc.* V. 49, p. 57 (LR49-1)]

Some experiences gained from construction in prestressed concrete, system Žežlj, erected in Slovenia (in Slavonian)

Sergej Bubnow, *Gradbeni Vestnik* (Ljubljana), V. 8, No. 47-50, 1956-57 pp. 143-150

Reviewed by J. J. Polivka

Prestressed concrete structures were introduced in Slovenia in 1954, and since then this material has been used for many commercial buildings. Author describes typical roof structure spanning 65 ft, consisting of I-shaped girders 16 ft on centers supporting continuous series of precast joists of inverted U-shape, 1½ in. thick and 7 in. deep, their width decreasing from 10 to 8¾ in. toward the top, thus providing, when assembled together, joints of triangular cross section, 2½ in. wide on the top. These joints were filled with special bituminous material, spread also on the surface as the lowest layer of a very reliable roofing. Joints on top of the girders are 4 in. wide. I-shaped girders with open web in the center part are 50 in. deep at the ridge and 40 in. at the supporting columns. Average ultimate strength of 0.2-in. wires was 225 kips per sq in. Process of testing materials is described. Average concrete strength was 8000 psi and slippage only 0.1 in. Cables were grouted with colloid cement and Lurgi plasticizer. Loading tests within elastic limits proved the accuracy of design. Comparison of costs shows that the prestressed concrete girders were 19 percent cheaper than ordinary reinforced concrete.

Properties of Concrete

Investigation of the influence of shape of concrete members on compressive strength (in German)

Kurt Walz, *Proceedings*, Deutscher Ausschuss für Stahlbeton (Berlin), V. 122, 1957, pp. 1-8

Reviewed by Patrick E. Murphy

Tests for influence of the shape of concrete specimens on compressive strength were made on a total of eight different cross sections. Results indicated very little shape effect, with the possible exception of the cylindrical cross section.

"Ytong"—Lightweight lime concrete (in Hebrew)

J. Karni, *In the Field of Building; Bulletin* No. 53, Technion—Israel Institute of Technology (Haifa), 1958, 12 pp.

Author's Summary

A brief review is given of the production process, followed by tables and diagrams summarizing the data on the volume weight, compressive strength, and absorptive capacity of the material (tests carried out on 15-cm cubes) and block. Statistical analysis showed a high degree of uniformity.

Finally, empirical formulas for the relationship between compressive strength (saturated) and volume weight (air dry) are given for block of different depth.

Structural properties of magnetite concrete

Jerome M. Raphael, *Proceedings*, ASCE, V. 84, ST 1, Jan. 1958, pp. 1511-1 to 1511-26

Magnetite concrete for a test reactor biological shield was made from combination of crushed magnetite aggregate and a low-alkali Type I cement. The elongated shape and high density of the crushed magnetite particles tended to cause harshness and segregation in mixing the concrete, but by paying careful attention to the proportion of fines and the composition of the cement paste in the mix, workable concrete mixes were achieved having densities ranging from 225 to 239 lb per cu ft. It proved important to prevent the cement paste from becoming too fluid, as this caused

excessive bleeding and segregation. Concrete of any required strength could be made, but for concretes having compressive strengths less than 4000 psi at 28 days, extra fines had to be added to the mix in place of cement to lend supporting power and lubricity to the paste.

Data are given on: design of the shield to resist large temperature differentials between the outer and inner faces; effect of volume changes; mix proportioning; densities; loss of weight on drying; bleeding; strength; modulus of elasticity and Poisson's ratio; shrinkage and coefficient of thermal expansion; and radiation shielding.

Structural Research

Effect of compressive stress on the dynamic modulus of concrete

R. H. Elvery and M. Furst, *Magazine of Concrete Research* (London), V. 9, No. 27, Nov. 1957, pp. 145-150

Author's Summary

A small reduction of modulus occurs when concrete is loaded but no subsequent reductions occur while the load is maintained for periods up to 6 months. A similar reduction also occurs on releasing the load and with subsequent loading cycles. This effect, which is independent of the age of the concrete after 14 days, is approximately proportional to the applied stress and becomes successively smaller for subsequent loading cycles. The reduction in dynamic modulus is always small, being about 5 percent or less.

Physical significance of tensile strength of concrete in relation to the modulus of rupture (in Russian)

K. A. Mal'tzov, *Beton i Zhelezobeton* (Moscow), No. 3, Mar. 1958, pp. 107-111

Reviewed by D. Watstein

The author studied plain concrete beams under a concentrated load and observed that "microcracks" appear at flexural stresses equal to tensile strengths obtained in axial tensile tests. The load causing microcracks applied for the first time in static tests does not cause failure. Failure is finally caused when the load is considerably increased.

Plane sections reportedly remained plane up to stresses at which the first microcrack was observed. Above this stress, the strain at the critical section where failure ultimately occurred increased quite rapidly, while at the adjacent sections the strains either remained constant or diminished.

The author interprets the results as indicating that the extensibility of concrete is the same in both axial tensile and flexural tests. He attributes the apparent increase in modulus of rupture to the ability of a flexural specimen to absorb considerable elastic energy following the formation of a microcrack.

The author offers the following equation for estimating the modulus of rupture:

$$R_m = \left(\frac{13.4}{h} + 1\right) R_T$$

where R_m and R_T are the modulus of rupture and tensile strength, respectively, and h is the depth of the test prism, in cm.

Load distribution in bridge slabs (with special reference to transverse bending moments determined from tests on three prestressed concrete slabs)

R. E. Rowe, *Magazine of Concrete Research* (London), V. 9, No. 27, Nov. 1957, pp. 151-160

Author's Summary

Tests on three prestressed concrete slabs of varying span/breadth ratios have given data concerning the load distribution properties of such structural forms.

The results for the distribution of deflections and longitudinal and transverse bending moments under one, two, and four equal loads are given and compared with results given by the theoretical analysis. The effect of considering Poisson's ratio in the theoretical analysis is discussed.

The distribution of the transverse bending moments, both in the transverse and longitudinal directions, is considered in detail.

Experiences with strain gage measurements in massive concrete structures (Erfahrungen mit Dehnungsmessungen im Massenbetonbau)

Rudolf Kuhn, *VDI Zeitschrift* (Düsseldorf), V. 99, No. 17, June 11, 1957, pp. 751-763

Reviewed by Aron L. Mirsky

An excellent article on measurements to evaluate the stress states in massive concrete members and structures. Author discusses the instrumentation involved: the temperature gage, the strain gage, the "null" (stress-

free comparison) gage, the force gage (as used to measure total force in a reinforcing bar); and then discusses the analysis of the data obtained to yield "pure" values of strain due to temperature, shrinkage, load, creep. Points made are well illustrated by references to investigations conducted on several power plants, among them the Jochenstein plant. Final portion of paper demonstrates (Fig. 55-62) marked discrepancies found between calculated and actual stresses at Jochenstein.

General

Pool and patio

Building Products, V. 4, No. 5, May 1958, pp. 31-81

A special report on residential and commercial swimming pools. Includes discussions of commercially available filtration systems, pumps, mechanical equipment and all structural types and materials available. Short descriptions of the various structural types commercially available include sizes, costs, package equipment, pool offers, etc. Addresses of suppliers are furnished as a source of further information on the various types.

A building collapses—why? (Un edificio se desploma .. ¿por qué?)

Construcción, V. 46, No. 6, June 1958, pp. 40-42, 86

A day-by-day account of the progressive differential settlement of the 11-story Sao Luis Rei apartment building in Rio de Janeiro leading up to the overturning and collapse of the structure Jan. 30, 1958. Reasons for the failure of this building and others in that city are analyzed. Included are recommended practices and a discussion of the flexibility of the codes of construction.

Protection and manipulation of electrical-resistance strain gages of the bonded-wire type for use in concrete, particularly for internal strain measurement

G. Hondros, *Magazine of Concrete Research* (London), V. 9, No. 27, Nov. 1957, pp. 173-180

Author's Summary

Describes techniques whereby electrical-resistance strain gages, of the bonded-wire type, can be used to measure strain at any point in a concrete or reinforced concrete member. Some observations are recorded on long-time stability of gages and instances are recorded where they have been successfully used to measure internal concrete strains and strains in reinforcing bars.

Investigations on building fires. Part V—Fire tests on structural elements

N. Davey and L. A. Ashton, Her Majesty's Stationery Office, London, (issued as Research Paper No. 12 of the National Building Studies), 1953, 274 pp., 15s.

The fifth in a series describing the results of investigations made by the Building Research Station (England) on fire resistance. Summarizes methods of testing used on structural elements, walls, partitions, columns, floors, and beams, and records the results of tests made during the years 1936 to 1946. Describes the testing equipment used and discusses the behavior of different materials and different structural systems under fire test.

Statistics manual

E. L. Crow, F. A. Davis, M. W. Maxfield, U. S. Naval Ordnance Test Station, China Lake, Calif. (available from Office of Technical Service, U. S. Department of Commerce), 1955, 288 pp., $6

Resulting from government sponsored research by the Naval Ordnance Test Station, this manual of statistical procedures is intended for use by scientists and engineers. Provides a precise method of summarizing conclusions drawn from experiments already performed and a reliable method of predicting information which can be gained from a proposed experiment particularly useful in the planning of tests and experiments on a more rational basis.

Building loss possibilities from fire and natural hazards

National Board of Fire Underwriters, 1958, 22 pp.

Explains building loss possibilities, causes of such losses, and how to provide safeguards against them. Discusses factors affecting loss possibilities including type of building construction, enclosure of stairways, elevators, and other floor openings; subdivision of large areas, protection against fires in adjoining property, and fire resistant roof coverings. Should be of value to architects, engineers, builders, and owners.

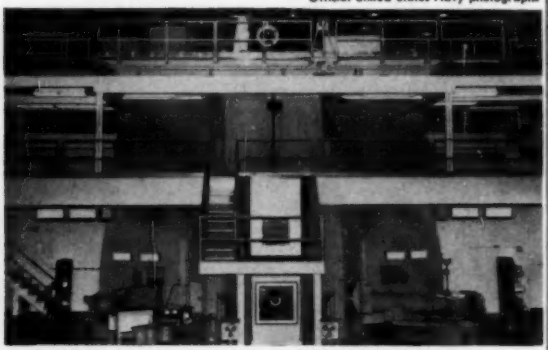
ACI News Letter

Vol. 30, No. 2 JOURNAL of the AMERICAN CONCRETE INSTITUTE August 1958

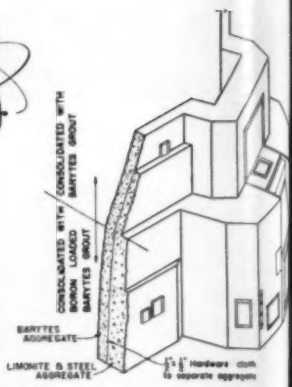
McGregor Center in concrete . . . p. 3

unique Prepakt wall shields nuclear research at Naval Laboratory

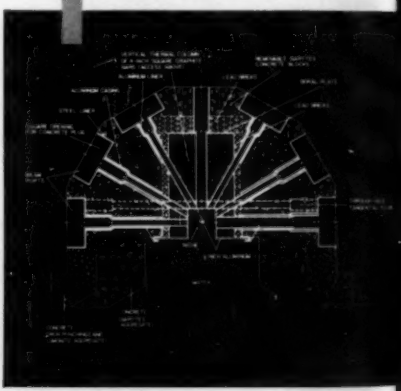
Official United States Navy photographs



Prepakt high-density concrete shield forms one end of Naval Research Laboratory "swimming pool" reactor. Behind shield reactor core is immersed in pool 40 ft. by 26 ft., 20½ ft. deep. Besides assuring safety from harmful radiation, Prepakt methods eliminated water leakage problems usually associated with difficult placement of conventional concrete.


CONSOLIDATED WITH BORON LOADED BARYTES GROUT
CONSOLIDATED WITH BARYTES GROUT
BARYTES AGGREGATE
LIMONITE & STEEL AGGREGATE

Prepakt concrete is made by consolidating pre placed aggregates with special grouts. For t concrete shield the aggregate nearest the co was barytes, for absorbing neutrons. Directly be hind, a ½-inch mesh screen separates the baryt from a blended coarse aggregate consisting o limonite and steel punchings. This section was t stop gamma rays. All aggregates were precis preplaced, then intruded simultaneously with grout containing boron frits and barytes sa When the grout level reached an elevation wh less shielding was needed, the expensive bor frits was withdrawn from the mix and a strai barite grout used to complete the placement.



Horizontal cross section through Prepakt shield illustrates nece sity for using concrete that assures complete embedment o beam ports and a tight bond to metal inserts.

The Naval Research Laboratory Reactor in Washington, D. C. features one of the most unusual concrete biological shields ever constructed. According to specifications, the shield was to be composed of four different mixes of high-density concrete cast monolithically without cold joints. The complexity of mix design and the necessity for precise, uniform placement made it advisable to call in shielding concrete specialists. INTRUSION-PREPAKT was selected because its experience and unique methods were ideally suited to the project.

Safety, economy and operational effectiveness were all considered in designing this shield. It was desirable that it be as thin as possible to hold the length of beam ports to a minimum and thus intensify the neutron flux at the ports' outer ends. At the same time the shield must be dense and leak-free to assure personnel of complete safety. PREPAKT® high-density concrete provided the economical solution and met all conditions with an ease and sureness that goes hand-in-hand with experience.

Patented methods and materials in the hands of skilled personnel permit INTRUSION-PREPAKT to take jobs like this in stride. Tight schedules and difficult operating conditions are the rule rather than the exception. Whether your requirements are for high-density concrete or more conventional construction services, you will save time and money by calling INTRUSION-PREPAKT, INC., Room 568-0, Union Commerce Bldg., Cleveland 14, Ohio. European Division, Zurbaran 14, Madrid, Spain. In Canada, INTRUSION-PREPAKT, LTD., 159 Bay Street, Toronto, Ontario.

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ACI

News Letter Contents

August 1958

On the Cover—McGregor Memorial Conference Center on the Wayne State University Campus, Detroit, is one of the unique concrete structures which will attract attention of ACI members attending the 11th Regional Meeting, October 27-29. The building is one of several recently erected in a civic rebuilding and beautification program. Construction features folded plate concrete roof members, concrete columns, interior walls of concrete masonry. Designed by Minoru Yamasaki of Yamasaki, Leinweber and Associates, Birmingham, Mich.

—*Photograph courtesy Wayne State University*

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Headquarters dedication highlights 1958 regional meeting in Detroit

October 27-29

Dedication of the recently completed ACI headquarters building on October 29 will follow technical program sessions of the 11th ACI regional meeting slated for the Statler Hilton Hotel in Detroit, October 27 and 28. The increasingly international scope of Institute activities is reflected in a program which brings Prof. A. M. Haas from Delft, Netherlands, to discuss the relation of form to structural design at the meeting's opening session on Monday morning. Georg Wastlund, director of the Cement and Concrete Institute, Stockholm, Sweden, will appear at the building code session Tuesday, speaking on high strength steel and cracking in reinforced concrete members. A report on Russian progress in concrete technology will be presented by J. D. Piper, Portland Cement Association, Chicago, and Walter H. Price, U. S. Bureau of Reclamation, Denver, who recently returned from a tour of the Soviet Union.

Other concrete experts, from the Detroit area and elsewhere throughout the United States, will contribute stimulating variety to a program reporting on structural research, design, materials, and construction problems. Concrete shell roof construction, and production of precast panels are scheduled for attention along with a comparative cost study of blast resistant design. Raymond C. Reese, consulting engineer, Toledo, Ohio, chairman of ACI Committee 318, will report on forthcoming changes in the ACI Building Code.

An entire technical session will be given to the ACI building, with architect, structural engineer, and general contractor each reporting his own point of view. A color film will show precasting and erection of the intricately folded concrete roof of the Institute's new home. Designed as a showpiece in concrete as well as a functional center for ACI's

DESTINATION DETROIT . . .

ACI members at the 1958 regional meeting in the motor city, October 27-29, will see auto plants, steel mills, and skyscrapers, flanking the world's busiest river. A center of culture as well as industry, Detroit has 11 museums of art, science, and history. Nearby are world-famous Greenfield Village and the Henry Ford Museum, born of Henry Ford's imaginative interest in our American heritage, displaying historically

international operations, the new headquarters is the culmination of years of planning and fund raising effort on the part of the ACI Building Committee, led by the late Henry L. Kennedy until his untimely death last year.

Following inspection tours of headquarters by regional meeting visitors, A. Allan Bates, Portland Cement Association, Chicago, will preside at a building dedication luncheon, Wednesday, October 29. Dr. Bates, now chairman of the Building Committee, will introduce committee members and recognize the many contributions which made the new headquarters possible.

In addition to the five technical sessions (complete program on following pages), the 3-day program includes a social hour, and a ladies' program of activity, plans for which will be announced next month. J. Gardner Martin, Portland Cement Association, Lansing, Mich., is general chairman of the regional meeting, and Prof. Elihu Geer, University of Detroit, heads the planning group for the technical program.

MONDAY MORNING, OCTOBER 27

Design Session—President Douglas McHenry, *Chairman*

Introductory Remarks—President McHenry

Russian Progress in Concrete Technology

J. D. Piper, Portland Cement Association, Chicago, and Walter H. Price, U. S. Bureau of Reclamation, Denver

Relation of Form to Structural Design

A. M. Haas, Institute of Technology, Delft, Netherlands

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significant developments in American science, agriculture, industry, music, and statesmanship. Cranbrook Institute, widely known center of art and science education, is housed in buildings of exceptional architectural beauty just north of Detroit, in rolling Bloomfield Hills.

Downtown shopping is concentrated in an area eight blocks by four, holding 800 department and specialty stores. October promises pleasant and moderate weather

Strength of Concrete Under Combined Stresses
K. S. Pister and Boris Bresler, University of California, Berkeley

Analysis of Shear Test Data (work of ACI Committee 326)
I. M. Viest, AASHO Road Test, Ottawa, Ill.

Comparative Cost of Blast Resistant Design
Warren Yee, Smith, Hinchman, and Grylls, Detroit

MONDAY AFTERNOON, OCTOBER 27

Construction Session—Ben Maibach, Jr., *Chairman*

Concrete Shell Roof Construction Problems
O. G. Sharrar, O. W. Burke Co., Detroit

Precast Sandwich Panels for Shopping Center
To be presented by a representative of Marietta Concrete Corp., Marietta, Ohio

Design Problems with Precast Concrete Panels and Floor and Roof Systems
Victor Leabu, Giffels and Rossetti, Detroit

Concrete on Formwork
Harry Ellsberg, Giffels and Rossetti, Detroit, Chairman ACI Committee 622

TUESDAY MORNING, OCTOBER 28

Building Code Session—Raymond C. Reese, *Chairman*

High Strength Steel and Cracking in Reinforced Concrete Members
Georg Wastlund, Cement and Concrete Institute, Stockholm, Sweden

What's Coming in ACI Building Code Revisions
Raymond C. Reese, consulting engineer, Toledo, Ohio

Footings
Paul Rogers, consulting engineer, Chicago

Detroit Regional Meeting Committee

J. Gardner Martin heads the Detroit regional meeting committee with Warner Ohman acting as secretary; both are with Portland Cement Association, Lansing. John W. Winkworth, Winkworth Fuel and Supply Co., Detroit, is treasurer of the committee and in charge of financial arrangements for the meeting.

Prof. Elihu Geer, University of Detroit, is technical program chairman and William T. Young, Sika Chemical Co., Detroit, is in charge of the ladies program. Charles Lyon, The Master Builders Co., Detroit, will handle publicity. Lloyd Cheney, Wayne State University, is in charge of educational contacts; W. G. Irmscher, Peerless Cement Corp., has taken over membership promotion; and registration will be supervised by William A. Maples, American Concrete Institute. The three latter chairmen are also from the Detroit area.

Creep and Deflection
Speaker to be announced

Flat Slabs and Flat Plates for Industrial Buildings
Alfred Zweig, Albert Kahn Associates, Detroit; Joseph DiStasio, consulting engineer, New York; J. Watts, Harley, Ellington and Day, Detroit

TUESDAY AFTERNOON, OCTOBER 28, CONCURRENT SESSIONS

Materials Session—Paul F. Rice, *Chairman*

Heavy Media Separation
David A. Killins, Killins Gravel Co., Ann Arbor, Mich.

Special Finishes and Colors in Precast Concrete Wall Panels
Will Saia, Precast Concrete Products Co., Royal Oak, Mich.

Courtesy of the Henry Ford Museum, Dearborn, Mich.

GREENFIELD VILLAGE . . .

Regional meeting visitors will have opportunities for varied industrial, historical, and cultural "sight-seeing" in and around Detroit. Preliminary plans for the ladies' program include a tour of Greenfield Village in nearby Dearborn. Here one finds a collection of landmarks, of actual buildings, in which history has been made. More than 100 historically significant buildings from every part of America—and several from England—are assembled in this 200-acre outdoor museum. Landscape features include the quaint Ackley covered bridge, built in 1832, once spanning Wheeling Creek in Pennsylvania. Only a few of these covered bridges can be found in the United States today. The box-like bridge sides gave strength to the structure and the covering protected the main beams from the deteriorating effects of bad weather

Efficiency of Stationary Tilting Mixers
John Winkworth, Winkworth Fuel and Supply Co., Detroit

Lightweight Slag Concrete
D. W. Lewis, National Slag Association, Washington, D.C.

ACI Building Session—William Krell, *Chairman*

Precast Roof—Film and Commentary
William Krell, Portland Cement Association, Detroit

Architectural Concepts
Minoru Yamasaki, Yamasaki and Leinweber, Birmingham, Mich.

Design of ACI Building
C. S. Whitney, Ammann and Whitney, Milwaukee and New York

Construction Features
John Strang, Pulte-Strange, Inc., Ferndale, Mich.

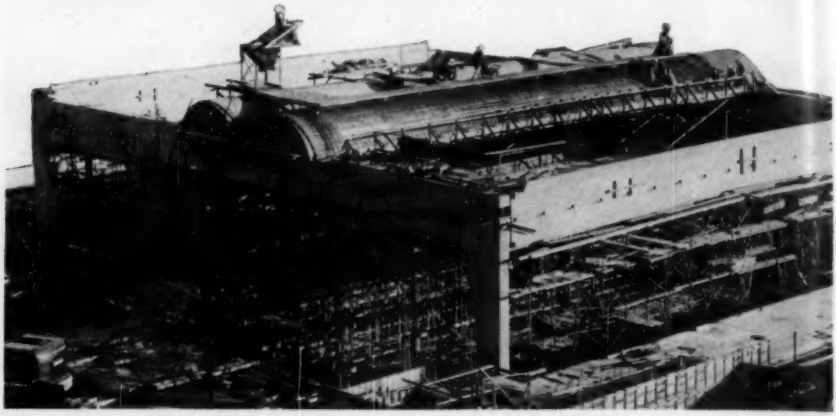
WEDNESDAY MORNING, OCTOBER 29

DEDICATION OF ACI HEADQUARTERS

A. Allan Bates, *Chairman*

Tour of ACI Headquarters Building, 9:30 to 12:00

Dedication Luncheon—Statler Hilton Hotel, 12:30



VAULTS ROOF GYMNASIUM

BARREL VAULTS span 104 ft 4 in. across the new gymnasium for the University of San Francisco. The vaults shown above during construction are 3½ in. thick and 18 ft wide, with an inside radius of 9 ft.

The ends of the barrel are open to allow fenestration and the area adjacent to the supports is designed as an arch to permit this opening. Except for a connection at the supports the barrels are not connected to each other. This permits each barrel to be cast and its shoring to be removed without affecting adjacent barrels. This method of design allows the contractor to reuse his forms and shores and the contractor used only two sets of forms for the job.

The barrels are supported on concrete bents (see picture) spanning 128 ft 4 in. A 1 x 10-in. tie bar connects the bottom of the bent columns to each other. This tie bar is placed just above the concrete floor, and after being fireproofed is covered by the gymnasium wood spring floor. This permits the tie bar to be tightened as the bent is unshored, to prevent spreading of the columns.

A concrete beam parallel to the barrel vaults combines with the bent to form a clerestory. Precast waffle panels, about 6 x 30 ft, will span from the bents or beam to the interior walls. The floor over the basement is of flat plate construction.

Bleachers are cast-in-place concrete. Footings are drilled-in-place piers supported on rock. The over-all dimension of the building is 179 x 165 ft; cost will be approximately $850,000.

Designed by Milton T. Pflueger, architect, the building is being built by Barrett Co., contractors. Structural engineer is ACI member Isadore Thompson. All three are from San Francisco.

New headquarters for Ford, Bacon and Davis

Ford, Bacon and Davis Construction Corp., subsidiary of the New York engineering firm of Ford, Bacon and Davis, Inc., has occupied new headquarters in Monroe, La. The firm planned and built the building with architectural assistance from Johns and Neal, architects of Monroe, La.

The 14,900 sq ft building was completed in 7 months from the time work started on architectural and engineering design. The building has a raised reinforced concrete floor slab, with a crawl space beneath. The roof is supported by concrete filled pipe columns, so that neither interior nor exterior walls are load bearing.

Positions and Projects

ACI Southern California Chapter elects officers

Ernst Maag, principal structural engineer, California Department of Public Works, Division of Architecture, Los Angeles, was elected president of the ACI Southern California Chapter at their meeting held at the Rodger Young Auditorium, Los Angeles, June 26.

J. L. Peterson, president of J. L. Peterson, Inc., Los Angeles, was elected vice-president.

Seven directors were elected: Hugh Barnes, Portland Cement Association; George C. Guibert, Super Concrete Emulsions; O. C. Struthers, Raymond International Inc.; Byron P. Weintz, Consolidated Rock Products Co.; and Emil R. Wohl of Wohl-Calhoun Co., all of Los Angeles; also Charles J. Pankow, Peter Kiewit Sons' Co., Arcadia; and Edward P. Ripley of General Concrete Products, Inc., Van Nuys.

Henry M. Layne, structural engineer, and William T. Wright, of Kistner, Wright and Wright, both of Los Angeles, were elected to a two-man policy committee to serve with the president.

Samuel Hobbs is secretary-treasurer of the chapter with headquarters in Los Angeles.

ACI technical committee appointments

Listed below are committee members who have recently accepted appointment to ACI technical committees. Included are *new* appointments only.

Committee—216, Fireproofing or Fire Protection of Structures

Robert R. Sheridan
Eastman Kodak Co.
Rochester, N. Y.

Committee 314, Rigid Frames for Buildings and Bridges

Sabri Sami
Alabama Polytechnic Institute
Auburn, Ala.

Committee 321, Design of Reinforced Concrete Slabs—Joint ACI-ASCE

Paul Rogers
Consulting Engineer
Chicago, Ill.

A. C. Scordelis
University of California
Berkeley, Calif.

Committee 401, Specification for Structural Concrete

Julius Adler
Philadelphia, Pa.

Committee 714, Concrete Bins and Silos

André Reimbert
Consulting Engineer
Paris, France

Committee 716, High Pressure Steam Curing

William H. Kuenning
Portland Cement Association
Chicago, Ill.

Wiedyke returns to ACI staff

Robert G. Wiedyke, who just completed 21 months of service with the 538th Engineer Battalion (Construction) at Fort Knox, Ky., is now back at ACI headquarters as an associate editor.

Mr. Wiedyke is a civil engineering graduate of Wayne State University, Detroit. While in the army he served as a design draftsman in a unit engaged in building roads and bridges on the army post.

Kerekes joins California division of architecture

George J. Kerekes is now associated with the California Department of Public Works, Division of Architecture, San Francisco, as senior structural engineer. Mr. Kerekes, formerly with Bechtel Corp., San Francisco, has been an ACI member since 1946 and is currently a member of Committee 314, Rigid Frames for Buildings and Bridges.

Shilstone establishes scholarship fund at Tulane

Tulane University and Loyola University, both in New Orleans, have each received a gift of $5000 from Shilstone Testing Laboratory, New Orleans. The gifts will establish a scholarship fund for chemistry studies to be known as the Dr. Herbert M. Shilstone Graduate Scholarship.

Under terms of the gift, $1000 per year will be available for use of one or more students who wish to work toward an advanced degree in chemistry or chemical engineering. The total will be spread over a 5-year period.

The late Dr. Shilstone, in whose honor the scholarship was established, was a graduate of the Tulane school of medicine and a former faculty member of the medical school.

Swayze presented ASTM award of merit; Kesler and Chang receive Sanford E. Thompson Award

Myron A. Swayze, director of research, Lone Star Cement Corp., New York, was one of 12 leaders in the field of engineering materials honored during the American Society for Testing Materials 61st annual meeting at Boston in June.

Presentation of an award of merit was made to Mr. Swayze in recognition of longtime, valued service and support of ASTM administrative and technical activities, especially for constructive contributions to the work of Committee C-1 on Cement.

In addition to ASTM activities, Mr. Swayze is active in the American Concrete Institute, Portland Cement Association, and the American Petroleum Institute. He was a director of ACI for two periods, 1944-45, and 1948-50. Many of his technical papers have been published in the ACI JOURNAL. In 1942 he was an ACI Wason Medalist for noteworthy research reported in the paper "Early Concrete Volume Changes and Their Control." Active in Institute committee work for many years, he is currently a member of the executive group of Committee 115, Research.

Clyde E. Kesler, professor of theoretical and applied mechanics, University of Illinois, Urbana, and Tien S. Chang, associate professor of applied mechanics, Virginia Polytechnic Institute, Blacksburg, were presented the Sanford E. Thompson Award during the same ASTM sessions.

The award, established by ASTM Committee C-9 on Concrete and Concrete Aggregates, was made in recognition of their outstanding paper, presented at the 1956 annual meeting, entitled "Correlation of Sonic Properties of Concrete with Creep and Relaxation."

Both authors are well known to ACI JOURNAL readers. Their paper, "Fatigue Behavior of Reinforced Concrete Beams," appears on p. 245 of this issue.

Reminder to ACI Members

Institute members in the United States and its possessions recently received a business reply card requesting important information. The advertising and editorial departments of your ACI headquarters staff urgently need these facts for compilation as soon as possible. Each member is requested to fill in the card and return it to ACI headquarters at your earliest convenience. DO IT TODAY.

If you have misplaced the card, clip the coupon on p. 23 of this News Letter and fill in the information requested.

BRI schedules 2-day floor conference

Solutions to major problems in the installation and maintenance of resilient flooring will be the theme of the Building Research Institute conference to be held September 17 and 18 at the Sheraton-Park Hotel, Washington, D. C. The speaker's roster includes J. J. Shidler, Portland Cement Association, Chicago, who will base his talk on concrete construction as a base for flooring. An ACI member since 1947, Mr. Shidler is currently serving on ACI Committee 613, Recommended Practice for Proportioning Concrete Mixes, and Committee 623, Specifications and Practices for Foamed Concretes.

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Return this order form together with your remittance—to avoid the added expense of billing—and your certificate will be mailed to you. Please allow 90 days for preparation of the certificate.

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(Print or type)

Signature_____

Robert F. Blanks

Robert F. Blanks, past president of the American Concrete Institute, died July 14, 1958. Mr. Blanks, vice-president and general manager, Great Western Aggregates, Inc., Denver, leaves an impressive record of outstanding accomplishments in the fields of civil engineering, research, and concrete.

Following graduation from Kansas State College in 1924, he engaged in private irrigation engineering in Idaho prior to joining the U. S. Bureau of Reclamation, Denver, in 1925. He ultimately became chief of the Research and Geology Division of the Bureau at Denver, serving in that capacity until 1951 when he joined Great Western Aggregates. While associated with the bureau, Mr. Blanks was active in the development of mass concrete methods for Hoover and other large reclamation dams, and was responsible for many technical advances in the field of concrete. The engineering and geological laboratory under his direction broadened its fields to become one of the largest laboratories of its kind in the world.

Active in international professional affairs, Mr. Blanks served on subcommittees of the World Power Conference and the International Commission on Large Dams. In 1946 he was appointed a member of the faculty of the graduate school of engineering of Harvard University and over a period of 2 years presented a series of lectures in connection with Harvard's graduate course on concrete technology.

Mr. Blanks received the honorary doctor of science degree from his alma mater in 1949. The Thomas Fitch Rowland prize of the American Society of Civil Engineers was given jointly to Mr. Blanks and the late Harmon S. Meissner, also of the Bureau of Reclamation, in 1947.

President of the American Concrete Institute during 1948, Mr. Blanks served on the ACI Board of Direction and on many Institute technical committees as well as the ACI Building Fund Committee. He frequently contributed to JOURNAL literature, and was coauthor with the late Henry L. Kennedy of a text book, *The Technology of Cement and Concrete, V. 1—Concrete Materials*, published in 1955.

At the time of his death Mr. Blanks was a member of ACI Committee 207, Properties of Mass Concrete, and Committee 213, Properties of Lightweight Aggregates and Lightweight Aggregate Concrete. He was also chairman of the Reinforced Concrete Research Council and active on several committees of the American Society for Testing Materials and the American Society of Civil Engineers.

Raymond and McKee form overseas organization

Raymond International Inc., New York, and Arthur G. McKee and Co., Cleveland, have organized a working association to completely design, engineer, and construct petroleum refineries and chemical plants overseas and outside North America on a single contract basis.

By joining forces, the two companies offer the petroleum and chemical industries the advantages of their combined international background and extensive engineering and construction experience.

Concrete construction scheduled in Alaska

The U. S. Army Engineer District, Anchorage, Alaska, has announced several construction projects scheduled in that area. Work will start shortly on two heavy support shops for NIKE installations in the Anchorage and Fairbanks areas. Contracts for the projects were awarded during the winter to Lease Co., Inc., Seattle, for the Anchorage job and to Commercial Construction Co., Fairbanks, for the Fairbanks project. Construction consists of a concrete frame building with concrete floor slab and pitched corrugated aluminum roof. Both projects are to be completed by Oct. 31, 1958.

Lease Co. has also been awarded a contract for construction of a complete modern airmen's service club at Elmendorf Air Force Base. The two-story building of concrete

and steel frame construction will contain approximately 27,800 sq ft of floor space.

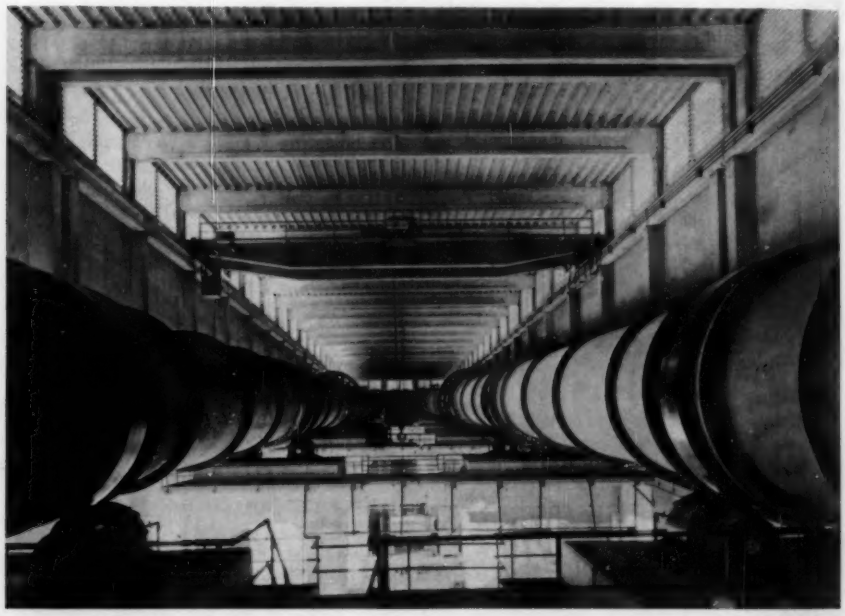
Bids on the construction of a new officers' open mess at Elmendorf Air Force Base were opened June 3. Basic schedule calls for a building of tilt-up concrete and concrete block masonry, approximately 23,000 sq ft.

Bids have also been opened on a vehicle maintenance shop and storage building to be built at the Army post of Wildwood Station, structure to be of steel and concrete frame with either tilt-up or concrete unit masonry panels.

Stein and Plummer elected by specifications group

J. Stewart Stein, member of the firm of Walter H. Sobel-J. Stewart Stein, Chicago, was recently elected national president of the Construction Specifications Institute whose headquarters are in Washington, D. C.

Harry C. Plummer, Structural Clay Products Institute, Washington, D. C., was elected secretary-treasurer. Both Mr. Stein and Mr. Plummer are ACI members.



FLORIDA DIVISION of General Portland Cement Co. will soon begin production at its new 2,500,000 bbl capacity plant 20 miles west of Miami. The plant occupies a 2400-acre site from which oolitic and coral limestone will be taken for cement manufacture. This development plus expansion of the company's Tampa plant, increases its capacity in the Florida market to 6,500,000 bbl.

The kiln building pictured houses two 425-ft kilns, is one of the largest buildings ever erected by the slip form method, with walls and columns cast integrally. Taking advantage of the latest techniques in concrete construction, the plant features prestressed concrete beams, precast columns and wall sections, as well as slip-form construction. The entire facility was designed and built by the Austin Co., in cooperation with the owner's engineers.

Louis G. Puls named chief designing engineer

Louis G. Puls has been named chief designing engineer of the Bureau of Reclamation at Denver. He succeeds Kenneth B. Keener who retired April 30 after 47 years of service with the bureau.

Prior to his present appointment, Mr. Puls was assistant chief designing engineer, serving in that capacity since 1956. He was head of the concrete dams section from 1952 to 1956, supervising many designs, including Glen Canyon, Flaming Gorge, and Monticello dams.

Peltier guest speaker at military engineer dinner

Rear Adm. Eugene J. Peltier, chief of the Civil Engineer Corps and Bureau of Yards and Docks, U. S. Navy, was honor guest speaker at the annual military engineer dinner held in connection with the 38th annual meeting in May of the Society of American Military Engineers. An Institute member, Admiral Peltier is a member of ACI Committee 613, Recommended Practice for Proportioning Concrete Mixes.

Major Gen. Emerson C. Itschner, chief of engineers, U. S. Army, was installed as president succeeding Rear Adm. H. Arnold Karo.

Turner awarded Idlewild maintenance base contract

Turner Construction Co., New York, has been named general contractor for the $4,500,000 aircraft maintenance base being built by Seaboard and Western Airlines at New York International Airport.

The 13-acre facility will include a structure covering approximately 140,000 sq ft, including approximately 90,000 sq ft of hangar area and a lean-to area of approximately 50,000 sq ft. Paved ramp will cover 230,000 sq ft; parking area, approximately 84,000 sq ft. Plans call for completion of the base in August, 1959.

Architects for the project are Kahn and Jacobs, Roy S. Bent, associate.

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LABYRINTH AVAILABLE IN 2, 3 or 4 rib.

ON YOUR CONSTRUCTION:

1. Consider the investment in design, materials and labor (to mention a few).
2. Then consider how important safe, secure *watertight* concrete joints are.
3. Thorough watertightness *can* be secured by installing Labyrinth Waterstops — a dividend that makes the low initial cost of the product insignificant when compared to your total investment—and one that insures watertight concrete joints for years!

- Corrugated ribs grip concrete, insure an everlasting bond between joints.
- Finest polyvinyl plastic resists chemical action, aging, severe weather.
- Takes just seconds to nail to form . . . easy to cut and splice on location (prefabricated fittings available).
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If your aim is to stop water seepage, *stop it effectively with Water Seals' Waterstops!*

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Prestressed concrete saves time

The Press Building is located on East Ninth Street and Lakeside Avenue in Cleveland. Total cost of construction is $10,000,000.

Design and Construction:
The Austin Company, Cleveland

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Placement of Prestressed Beams and Girders—The beams are 2′8″ high by 30′ long and weigh five tons each; the girders are 4′6″ high by 30′ long and weigh 11 tons each. These prestressed members were designed to support the unusually heavy loads of 400 pounds per sq. ft.; the live load is 350 pounds per sq. ft. The builders used lightweight concrete prestressed with USS Super-Tens Stress-Relieved Wire for economy and to speed enclosure of the structure before the advance of winter weather.

in the new Cleveland Press Building

Beams and girders are prestressed with extra-straight USS Super-Tens Stress-Relieved Wire

THIS IS THE NEW HOME of Ohio's largest daily circulation paper, the Cleveland *Press*. It's obviously a new kind of newspaper building, built with modern construction ideas and methods—built with *prestressed concrete.*

Construction time cut. The *Press* Building is scheduled for completion early this fall. The use of prestressed concrete helped accelerate the construction schedule before the winter months. Beams and girders were prestressed at a prestressing plant, then shipped to the job site for immediate installation.

Progressive designers are using prestressed concrete in many ways—in bridges, office buildings, factories, warehouses, and schools. In design, prestressed concrete makes possible thinner sections, lower depth-to-length ratios, reduction in dead weight, and permits longer cantilevering. Prestressed concrete is easier to erect because there is no form work to do on the job.

American Welded Wire Fabric—This all-concrete building is also being strengthened with another famous construction product of American Steel & Wire—USS American Welded Wire Fabric Reinforcement. The precast floor channels, cast-in-place roof and floor slabs, and the slabs on grade were all reinforced with extra-strong, welded wire fabric.

For more information, call or write American Steel & Wire, Rockefeller Building, Cleveland 13, Ohio.

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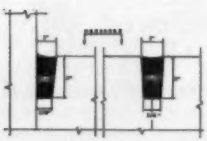

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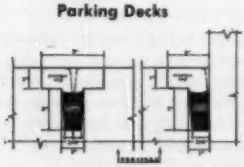

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Who's Who This Month

ACI Committee 612

The report of ACI Committee 612 on "Curing Concrete" appears on p. 161. G. E. Burnett, Bureau of Reclamation, Denver, is committee chairman.

Committee members include: Edward E. Bauer, University of Illinois, Urbana; H. F. Clemmer, Government of the District of Columbia, Washington, D. C.; T. E. Howell, Caterpillar Tractor Co., Peoria, Ill., and W. R. Johnson, Corps of Engineers, Portland, Ore.

Also serving on this committee are: Bryant Mather, Waterways Experiment Station, Jackson, Miss.; Francis A. McAdam, Huron Portland Cement Co., Detroit; Mark Morris, Iowa State Highway Commission, Ames; F. V. Reagel, Missouri State Highway Commission and George Werner, Bureau of Public Roads, Washington, D. C.

ACI Committee 622

"Pressures on Formwork" on p. 173 of this issue was reported by ACI Committee 622 with Harry Ellsberg, Giffels and Rossetti, Detroit, serving as chairman of the group.

Members of the 20-man committee which prepared the report are: John Banker, Portland Cement Association, Chicago; Frank H. Beinhauer, J. L. Simmons Co., Inc., Decatur, Ill.; H. P. Cerutti, Blaw-Knox Co., Pittsburgh; Edgardo Contini, Victor Gruen Associates, Beverly Hills, Calif.; N. L. Doe, Turner Construction Co., New York; and Jacob Feld, consulting engineer, New York.

Others on this committee are: David E. Fleming, structural engineer, Lakewood, Colo.; Vance J. Gray, associate, Raymond C. Reese, Toledo, Ohio; Robert C. Johnson, Siesel Construction Co., Milwaukee; J. C. McCloskey, Jr., McCloskey and Co., Philadelphia; Verne O. McClurg of McClurg, Shoemaker and McClurg, Chicago; and Donald R. Peirce, Ammann and Whitney, New York.

Also contributing their time and effort to Committee 622 are: A. H. Pilling, Richmond Screw Anchor Co., Inc., Brooklyn, N. Y.; Joseph R. Proctor, Jr., Dravo Corp., Warsaw, Ky.; H. R. Puffer, Michigan State Highway Department, Lansing; and O. G. Sharrar, O. W. Burke Co., Detroit.

Remainder of the committee includes: M. L. Stephens, Dixie Form and Steel Co., San Antonio; P. R. Stratton, Hydro-Electric Power Commission of Ontario, Toronto; William R. Waugh, Office, Chief of Engineers, Department of the Army, Washington, D. C.; and William H. Wolf, Bureau of Reclamation, Denver, Colo.

Gene M. Nordby

"Fatigue of Concrete—A Review of Research," p. 191, prepared by Gene M. Nordby, formerly program director for engineering sciences, National Science Foundation, Washington, D. C., was presented at the Chicago ACI convention in February as the introduction to a symposium on fatigue.

On August 1, Mr. Nordby assumed new duties on the faculty of the University of Arizona, Tucson, as head of the civil engineering department.

Mr. Nordby graduated from Oregon State College in 1948 and received his MS and PhD degrees from the University of Minnesota. Before joining the foundation in 1956 as an engineer, he was a member of the civil engineering faculty at Purdue University and at the University of Colorado.

Mr. Nordby is chairman of ACI Committee 215, Fatigue of Concrete, and a member of ACI Committee 115, Research, as well as ACI-ASCE Committee 323, Prestressed Reinforced Concrete. He is a member of the Reinforced Concrete Research Council.

A paper "Fatigue and Static Tests of Steel Strand Prestressed Beams of Expanded Shale Concrete and Conventional Concrete" prepared by Mr. Nordby and William J. Venuti, San Jose State College, San Jose, Calif., appeared in the August, 1957, issue of the ACI JOURNAL.

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Sponge Rubber CEMENTONE®

High quality blown sponge rubber, uniform in thickness and density. Neutral gray color blends well with concrete.

advantages

1. **Blends with the color of concrete**
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recommended uses . . . For use in concrete structures where utmost resilience, non-extrusion and/or inconspicuous joints are desired. Ideal for use in tilt-up and bridge construction.

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John W. Murdock and Clyde E. Kesler

"Effect of Range of Stress on Fatigue Strength of Plain Concrete Beams," p. 221, by John W. Murdock and Clyde E. Kesler, was also presented at the 1958 convention as part of the fatigue symposium sponsored by ACI Committee 215.

Mr. Murdock is presently an instructor in theoretical and applied mechanics at the University of Illinois, Urbana. He graduated from Rose Polytechnic Institute in 1947 and served in the Civil Engineers Corps, United States Navy, until 1953. Subsequently, he worked for the design and construction sections, bridge division, Department of Highways of the District of Columbia.

He entered the University of Illinois in 1954 as a research assistant in theoretical and applied mechanics, studying on the fatigue behavior of plain concrete. He received his MS degree there in 1956.

Clyde E. Kesler, professor of theoretical and applied mechanics, University of Illinois, has had many technical papers published describing research in fatigue, creep, and sonic behavior of concrete. His most recent contribution to the ACI JOURNAL, "Static and Fatigue Strength in Shear of Beams with Tensile Reinforcement" coauthored with Tien S. Chang, appeared in June, 1958.

Professor Kesler graduated from the University of Illinois and has served as a consultant on industrial problems and research. He is a member of many honorary and professional groups including ACI, American Society of Civil Engineers, and ASTM.

An ACI member since 1947 he now serves on the Technical Activities Committee. Professor Kesler is also a member of Committee 209, Volume Change and Plastic Flow in Concrete, and a member and former chairman of Committee 215, Fatigue of Concrete.

John T. McCall

Third paper of the fatigue series is "Probability of Fatigue Failure of Plain Concrete," p. 233, by John T. McCall, associate professor in the department of applied mechanics at Michigan State University, East Lansing. He graduated from the Michigan College of Mining and Technology in 1947 and received

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- *Heavy Steel-Aggregate Concrete*—Gives consideration to various mix proportions for heavy concrete and a proportioning procedure for concrete of given strength and density.
- *Properties of Heavy Concrete Made with Barite Aggregates*—Presents data on test results on barite (barium sulfate) in conventional and prepacked concrete where high density is desired.
- *Magnetite Iron Ore Concrete for Nuclear Shielding*—Physical properties and costs of magnetite ore concrete are compared to those of other concretes.
- *Proportioning of Mixes for Steel Coarse Aggregate and Limonite and Magnetite Matrix Heavy Concretes*—A discussion of structural concrete utilizing limonite and magnetite ores as fine aggregate and graded steel scrap as coarse aggregate.

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his MS from Michigan State University in 1951 and his PhD from Purdue University in 1956. During World War II, Professor McCall served as an officer in the United States Marine Corps.

Joining the faculty at Michigan State in 1947, shortly after graduation, Professor McCall has engaged primarily in teaching and research in the field of plain concrete.

A member of several professional societies as well as ACI, he is now serving on ACI Committee 215, Fatigue of Concrete.

Tien S. Chang and Clyde E. Kesler

Coauthors Chang and Kesler make an early return to the ACI JOURNAL pages with their paper "Fatigue Behavior of Reinforced Concrete Beams" appearing on p. 245. Another joint paper contributed by these authors, "Static and Fatigue Strength in Shear of Beams with Tensile Reinforcement," was published in the June 1958 JOURNAL.

Tien S. Chang is a graduate of the University of Illinois where he spent the last 5 years on experimental and theoretical studies of viscoelasticity, fatigue and creep of concrete, and rheology. He is now an associate professor in the department of applied mechanics at Virginia Polytechnic Institute, Blacksburg, where he is actively engaged in the research of sonic testing, supersonic compressors, and magnetohydrodynamics. Professor Chang is a member of ACI, ASTM, and other professional societies.

Professors Chang and Kesler received the American Society for Testing Materials Sanford E. Thompson Award in June in recognition of their outstanding paper—"Correlation of Sonic Properties of Concrete with Creep and Relaxation"—presented at the 1956 ASTM annual meeting. Biographical data for Professor Kesler appear above in connection with his joint paper with John W. Murdock.

Thomas E. Stelson and John N. Cernica

"Fatigue Properties of Concrete Beams" on p. 255 is the joint presentation of Thomas E. Stelson and John N. Cernica, and concludes the symposium on fatigue.

Thomas E. Stelson, associate professor and acting head, department of civil engineering, Carnegie Institute of Technology, Pittsburgh, has been a member of the teaching staff there since 1952. Graduating from Carnegie Tech in 1949, he continued his studies receiving his DSc degree in 1952. Professor Stelson has directed several research studies on the behavior of concrete and concrete structures.

A member of a number of professional and honor societies, as well as ACI, Professor Stelson is currently a member of ACI Committee 115, Research, and Committee 215, Fatigue of Concrete.

John N. Cernica is assistant professor of civil engineering at Youngstown University, Youngstown, Ohio. Graduating from Youngstown University in 1954, he continued his studies at Carnegie Institute of Technology, receiving his PhD in 1957.

His present research activity involves the study of plastic flow in prestressed concrete, a project being conducted at the Youngstown University civil engineering laboratories.

James E. Backstrom, Richard W. Burrows, Richard C. Mielenz, Vladimir E. Wolkodoff, and Harry L. Flack

Messrs. Backstrom, Burrows, Mielenz, and Wolkodoff collaborated in the preparation of Part 2, "Influence of Type and Amount of Air-Entraining Agent," of the four-part "Origin, Evolution, and Effects of the Air Void System in Concrete." Part 1 was published in the July 1958 ACI JOURNAL; Part 2 appears on p. 261.

James E. Backstrom is head of the materials, mixes, and durability section of the Concrete Laboratory Branch, U. S. Bureau of Reclamation Materials Laboratory, Denver. Mr. Backstrom was also a coauthor of Part 1 of this series of papers, and a complete biographical sketch will be found in the July "Who's Who."

Richard W. Burrows is design consultant for the Martin Co., Denver, contractor for the Titan intercontinental balistic missiles. A graduate of the Colorado School of Mines, he was first employed as a metallurgist by the Aluminum Co. of America, and subsequently served as radar technician in the Navy during World War II.

Mr. Burrows joined the engineering laboratories of the Bureau of Reclamation in

1946 and has written a number of papers on concrete durability. He formulated a chemical grout which was successfully employed in the sealing of leakage at Heart Butte Dam, N. D. He remained with the bureau more than a decade, until assuming his present duties.

Richard C. Mielenz, director of research, The Master Builders Co., Cleveland, also coauthored Part 1 of this series. Biographical information appeared in the July "Who's Who."

Vladimir E. Wolkodoff, is senior engineer and head, physical measurements group, research and development division, The Carborundum Co., Niagard Falls, N. Y. His work deals mainly with the investigation of refractories, refractory cements, super-refractories, the manufacturing of synthetic minerals by electro-furnacing, physical properties of abrasives, investigation of extremely hard minerals, and high-temperature microscopy. He has been with The Carborundum Co. since May 1956.

Prior to his present affiliation he was engineering petrographer in the engineering laboratories division of the Bureau of Re-

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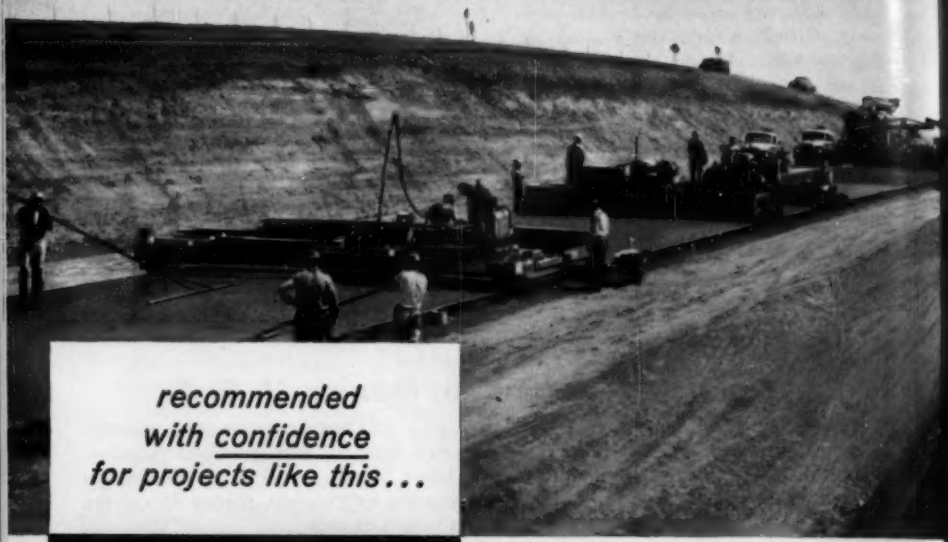

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clamation, Denver, from January, 1950 to May, 1956. It was during this time that he worked on research material for this four-part series on air void systems. His other main research interest then was the petrographic interpretation and study of strength and elastic properties of rock for engineering geology and related fields and behavior of aggregate in concrete.

Mr. Wolkodoff studied at Columbia College and University of Buffalo, receiving his MS degree in mineralogy in 1949. During 1948 and 1949 he was an instructor in mineralogy and crystallography at the University of Buffalo. He served with the U. S. Air Force during World War II.

Harry L. Flack was coauthor of Part 1 of this air void series. Mr. Flack graduated from the University of Colorado and is currently employed as engineer with the Bureau of Reclamation, Denver, engaged in research on the freezing and thawing durability of concrete.

After 2½ years with the 23rd Construction Battalion, U. S. Navy, he joined the Bureau of Reclamation in 1946 and has conducted various types of investigations and research into the properties of concrete and concrete materials. He has written several technical papers, the most recent appearing in *Proceedings*, ASTM, 1957, based on freezing and thawing resistance of concrete as affected by method of test.

Frederick P. Wiesinger

The author of "Design of Symmetrical Columns with Small Eccentricities in One or Two Directions," p. 273, is Frederick P. Wiesinger, assistant professor of civil engineering at the University of Illinois, Chicago Division.

After completing his studies as architect-engineer at the Royal Polytechnical University in Budapest, Hungary, in 1945, Professor Wiesinger practiced architecture during the post-war reconstruction period in Hungary till 1948. During 1949 he worked in Switzerland and in Paris. He came to the United States in 1950 and was engaged as a structural designer and project engineer with several engineering firms. He also taught refresher courses for the professional and structural engineers registration examinations at Illinois Institute of Technology and at the University of Illinois.

In addition to his teaching duties, Professor Wiesinger has maintained his own practice as consulting structural engineer since 1955, and is presently working toward a PhD at Northwestern University.

A member of a number of professional societies, Professor Wiesinger joined ACI in 1956 and is a member of Committee 322, Design of Structural Plain Concrete. In 1956 he married Mary-Jane Jensen, MD, who is a psychoanalyst (the family joke is that they both deal with stresses and strains.)

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Lepper joins University of Maryland Staff

Henry A. Lepper, Jr., formerly associate professor of civil engineering at Yale University, New Haven, has joined the teaching staff at University of Maryland, College Park, as professor in the civil engineering department.

British Columbia Cement donates park area

The British Columbia Cement Co., Vancouver, has donated 30 acres of waterfront park area for a future provincial park. In making this gift the company is following an already established precedent. Victoria's internationally acclaimed attraction, the Butchart Gardens, were developed by the wife of the first president of the company to cover the scars left by the old quarry at Tod Inlet. The Butchart Gardens are being perpetuated as a world famous tourist attraction under the management of Ian Ross, a long time Victoria resident and director of British Columbia Cement Co.

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Honor Roll

January 1—July 31, 1958

This month Blas Lamberti heads the Honor Roll with 14 new members to his credit. Presenting the results of both research and practice in the U. S. and throughout the world, the American Concrete Institute offers new members accessibility to a wealth of information in concrete and concrete construction that is unavailable elsewhere.

Blas Lamberti	14
Joseph J. Waddell	11
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Ernst Maag	6
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J. P. Thompson	5
Moises Bendahan	5
Celso A. Carbonell	4
P. E. Ellen	4
Carl H. Koontz	4
L. M. Legatski	4
Frank E. Legg, Jr.	4
Douglas McHenry	4
H. C. Pfannkuche	4
John B. Porter	4
Clarence A. Walkwitz	4
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Oscar J. Vago..... 2
Ellis S. Vieser..... 2
Jose Antonio Vila..... 2
Ivan A. Villamil..... 2
M. R. Vinayaka..... 2
Tsu-ming Yang..... 2

New Members

The Board of Direction approved 81 Individual applications, 5 Corporations, 27 Juniors, and 10 Students making a total of 123 new members. Considering losses due to resignations, deaths, and nonpayment of dues the total membership on July 1, 1958, was 9534.

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ARELLANO, ADOLFO R., Miramar, Habana, Cuba (Arch. & Engrg. Contrg. Colegio de Ingenieros Civiles de Cuba)

ARMENTO, WILLIAM J., Staten Island, N. Y. (Adjunct Prof., Polytechnic Inst.)

AUGUSTIN, HANS, Bancroft, Ont., Canada (Techn., Bicroft Uranium Mines)

BENNETTS, DEXTER HORTON, Gisborne, New Zealand (Tech. Officer, New Zealand Gov't)

BRADFORD, WILLIAM R., Miami, Fla. (Asst. Village Engr., Miami Shores Village)

BRCEK, JAROSLAV, Merida, Venezuela (C. E., Cons. Engr., Prof. of Bridges, Faculty of Civil Engrg., Universidad De Los Andes)

CHAW, ROMAN, Yonkers, N. Y. (Struct. Designer, Seelye, Stevenson, Value & Knecht)

CLAUSEN, C. F., Skokie, Ill. (Director, Mfg. Process Dept., PCA)

DAVIES, L. TRUMAN, Higham, Nr. Rochester, Kent, England (Gen. Mgr. & Tech. Dir., Colcrete Ltd.)

DEBBINK, JOHN P., Oconomowoc, Wis. (Pres., Design, Engrg. Sales, Metal Structures, Inc.)

DOSSEY, RAY, San Antonio, Texas (Engr. in Trng., Texas Stressed Concrete Corp.)

DRUBEL, RICHARD B., Midland, Mich. (Section Head, Bldg. Prod. Group, Coatings Tech. Service, Dow Chemical Co.)

ERMENKOV, S., San Diego, Calif. (Struct. Designer, Frank L. Hope, Arch.)

FELGER, OTIS A., Memphis, Tenn. (Vice-Pres., Lift Slab Management Corp., Southern Div.)

FIRTH, CHARLES R., San Francisco, Calif. (Engr., Gen. Design, Ben C. Gerwick, Inc.)

FOSTER, PAUL S. A., Seattle, Wash. (Struct. Engr. Bassetti & Morse, Archs.)

FULLANA, FRANCISCO, San Juan, Puerto Rico (Builder, engrg. firm (design) Fullana Constr. Co.)

FURSH, STEPHEN J., Fresno, Calif. (Constr. Supv., Calif. State Div. of Arch.)

GAMBINO, ROBERT J., Orange, Conn. (Struct. Engr., Engr. in Chge. Design of Structures in Concrete & Steel, Seelye, Stevenson, Value & Knecht)

GIACOMO, VENTURA GIAN, Brescia, Italy (Geometer)

GIBBE, KURT F., El. Paso, Texas (Chf. Chem., Southwestern Portland Cement Co.)

GOMEZ LESSEUR, GUSTAVO, Caracas, Venezuela (Civil Engrg., Design & Insp.)

GRAY, CHARLES R., Granite City, Ill. (Product Dev. Engr., Granco Products Co.)

GRIFFITH, KENNETH JOHN, Homebush, N. S. W., Australia (Chf. Design Engr., A. R. C. Eng. Co., Pty., Ltd.)

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MASTALIR, HOWARD W., Lincoln, Neb. (Struct. Engr., Schaumberg & Freeman, Archs.)

MCBRIDE, IAN F. B., Edmonton, Alta., Canada (Assoc. Struct. Engr., Rule, Wynn & Rule, Archs.)

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PUFFER, KAREL, Edmonton, Alta., Canada (Struct. Designer, Associated Engrg. Services, Ltd.)

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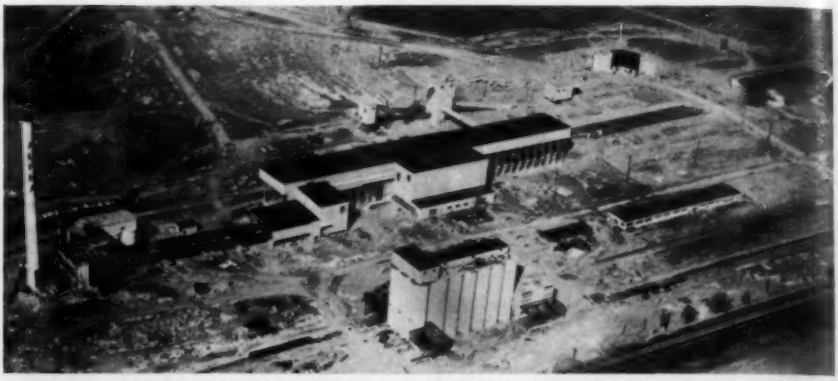

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RIZO, JOSE H,, Bogota, Colombia

Cutler promoted

M. H. Cutler, formerly chief structural engineer for Stone and Webster Engineering Corp., Boston, has been promoted to assistant engineering manager for the organization. Mr. Cutler joined Stone and Webster in 1923 and has done design and consulting work on power stations, industrial and chemical plants, and office and institutional buildings in many parts of this country and Canada.

ACI *Proceedings* in microfilm

ACI members and JOURNAL subscribers may obtain complete *Proceedings* volumes (including discussion) in microfilm. This is an economical means of maintaining a file of ACI material with the minimum storage space. *Proceedings* V. 46 (Sept. 1949 to June 1950) through *Proceedings* V. 53 (July 1956 to June 1957) are available.

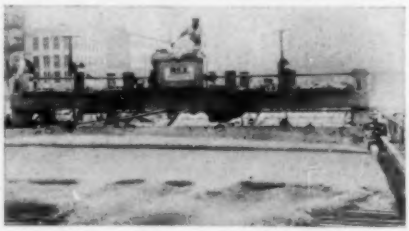
Proceedings for the last two volume years (V. 52 and V. 53) are available at $4.25 and $5.65 respectively. Volume 51 costs $4.40. For additional price information and orders, write University Microfilms, 313 N. First St., Ann Arbor, Mich.

Tools, Materials, Services

Under this heading note is made of producer literature and products of presumed technical interest to ACI users of tools, equipment, materials, accessories, and special services.

Concrete finisher for bridges

A concrete bridge finishing machine incorporating all the design and construction advantages of the standard Rex concrete road finishing machines is



produced on a made-to-order basis and can be converted into a standard road finishing machine with a minimum of labor.

Adjustable in both elevation of the large screed and in frame width, Rex powered-frame widening is available to provide instantaneous width changes where desired. Quick crown change attachment is available for easy transition work, as is hydraulic transportation rig complete with pneumatic tired wheels, hydraulic lifting pedestal, and hinged towing tongue for fast efficient movement from job sites.—Chain Belt Co., Milwaukee 1, Wis.

"Test-by-mail" service for highway material

The Mail-Test Division of the Haller Testing Laboratories is now equipped and staffed to speedily mass-test highway paving materials. Representative samples are mailed in preaddressed and stamped containers to the nearest laboratory branch where chemical and physical analyses are carried out. Reports are mailed back or called in, if a change is warranted, so remedial action can be taken rapidly.

Materials most commonly tested include portland cement concrete and bituminous paving products. Costs for physical and chemical analysis range from $15 to $35 per sample, making the service practical for private as well as for public works projects.—The Haller Testing Laboratories, Inc., 140 Cedar St., New York 6, N. Y.

Prestressed concrete film

The Florida Division of Food Machinery and Chemical Corp. has recently released a film entitled "Prestressed Concrete Methods" showing the basic methods involved in the prestressing of bridge beams, piling, double-T's and also the production of some of the largest prestressed girders ever cast. The film is available to anyone in the construction industry at no charge. Requests should be made at least two weeks in advance of showing date.—Food Machinery and Chemical Corp., Florida Division, Lakeland, Fla.

LOOKING AHEAD

Sept. 15, 1958—6th International Congress on Large Dams, New York, N. Y.

Sept. 21-25, 1958—Prestressed Concrete Institute, Fourth Annual Convention, Edgewater Beach Hotel, Chicago, Ill.

Oct. 13-17, 1958—American Society of Civil Engineers, Annual Convention, Statler Hotel, New York, N. Y.

Oct. 20-24, 1958—National Safety Congress and Exposition, Conrad Hilton and other hotels, Chicago, Ill.

Oct. 23-25, 1958—National Society of Professional Engineers, Fall Meeting, St. Francis Hotel, San Francisco, Calif.

Oct. 27-29, 1958—American Concrete Institute, Regional Meeting, Statler Hilton Hotel, Detroit, Mich.

Nov. 18-20, 1958—American Standards Association, Ninth National Conference on Standards, Hotel Roosevelt, New York, N. Y.

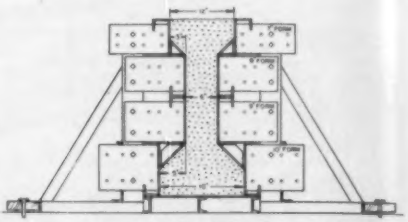
Dec. 7-10, 1958—American Institute of Chemical Engineers, Annual Meeting, Netherland Plaza Hotel, Cincinnati, Ohio

Feb. 23-27, 1959—American Concrete Institute, 55th Annual Convention, Statler Hilton Hotel, Los Angeles, Calif.

Adjustable steel forms

An adjustable steel form for the prestressed concrete industry has sidewalls made up of a series of panels which can be varied in several ways to produce the desired end result. The panels can be rearranged in sequence from top to bottom. They can be shifted horizontally, and when necessary, over-all height can be adjusted with addition of fillers.

Using a four-panel form with 7, 8, 9 and 10-in. panels, adjustments can be made to produce building beams, I-beams, and hollow box girders. The forms are normally furnished in 10-ft long panels. Panels can be manufactured in various heights to produce the beams most often cast by a given manufacturer. By varying the horizontal alignment of the panels on the



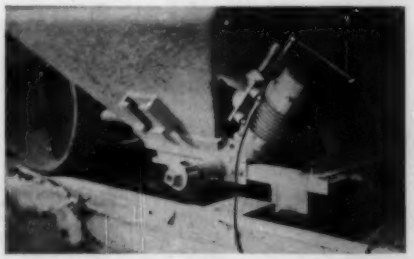
I BEAM GIRDER

frame, a prestress operator can also manufacture hollow box girders, building beams, and several other sizes of I-beams.

Company spokesman reports that while these forms are necessarily more expensive initially than a set of forms of comparable dimension designed for casting a single type of beam, they normally cost less than any two sets of "fixed" forms of comparable size.—Blaw-Knox Co., 300 Sixth Ave., Pittsburgh 22, Pa.

Electric vibrator to speed unloading

An electric vibrator designed to speed the unloading of dry cement, sand, and other types of powdered or granular materials uses a rotary eccentric. The "Vibramat" can be attached to the unloading hopper



of railroad freight cars or other transporting equipment. According to the manufacturer it sets up a concentrated vibration to loosen jammed or packed materials.

When used on concrete forms, the Vibramat is said to cause the wet concrete to settle smoothly into the form without hand rodding or pounding. Available with either a rigid or swinging base with 1/6, 1/3, or 1 hp motor. Rigid base model is suggested for use on concrete pipe forms where vibration in all directions is desired.—Engineered Equipment, Inc., Waterloo, Iowa

Lightweight aggregate for concrete

Completion of a plant at Ottawa, Ill., to manufacture lightweight aggregate, has been announced. The raw material is a special shale, ground, graded, and passed through long rotary dryer and kilns in which it is expanded. From these kilns come rounded and

tempered particles, which are said to have strength comparable to gravel but weigh only half as much.

Walter C. Voss, professor emeritus, architectural construction and materials, Massachusetts Institute of Technology, said of Materialite: "The extensive tests conducted under my supervision by the Armour Research Foundation of the Illinois Institute of Technology and Herman G. Protze, materials technologist of Boston, on the properties of concrete made with the lightweight aggregate produced by your company, convince me that it is the most unusual lightweight aggregate which has come to my attention. The fact that a 5-sack mix tested uniformly well over 3000 psi while the air-dry weight of the concrete was less than 85 lb per cu ft, is very unusual. Strengths of this order have been confined to the more dense gravel or stone concretes weighing 150 lb per cu ft. . . . "

Manufacturer also notes that Materialite differs from other aggregates available in the midwest because all crushing is done prior to expansion. It is said to have a protective coating, which requires less moisture and cement than aggregates crushed after expansion.—Material Service Corp., 300 West Washington St., Chicago 6, Ill.

Portable package vibrator control system for prestressed concrete

A portable package vibrator control system for the prestressed concrete industry consists of ten Cleveland RC-30 portable electric vibrators with mounting brackets and a portable mounted control panel center for all vibrator control. All ten vibrators are plugged into the portable control panel which is supplied by an outside power line.

The individual vibrators, which are secured to the form with two bolts, can be mounted and unmounted

in minutes according to the manufacturer; mounting brackets are welded to the forms and the vibrators are easily slipped in and out.

Vibrators are totally enclosed providing protection for rough usage. The RC-30 weighs 48 lb and will develop up to 1100 lb of impact at 3600 vibrations per min. Smaller and larger electric vibrators are available for prestressed concrete applications.—The Cleveland Vibrator Co., 2828 Clinton Ave., Cleveland 13, Ohio

No more algebraic formulas or calculations to make. Simply locate the table covering the member you are designing, apply span and load requirements, and then *read off directly* concrete dimensions and reinforcing steel data. Follows the latest codes and practices. Send check or money order for your copy, today.

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Device for sizing and desliming sand

The Wemco Sandclone uses the wet cyclone to separate sand efficiently into desired size fractions. The hydrocyclone type classifier utilizes centrifugal force set up within a cone to speed up this separation, according to the manufacturer.

Designed specifically for use in the fine aggregate industry, the Sandclone is particularly useful in the gradation of finer size fractions and in the separation of fine sands from slimes and clays. Fractions from No. 4 to No. 200 mesh are normally handled; a wide range of gradation requirements may be met with a minimum of adjustments. —*Western Machinery Co., 650 Fifth St., San Francisco 7 Calif.*

"Doubleheader" horizontal cylinder capper

By producing acceptably plane and parallel caps of identical thickness, with no heavy overhang around

the edges, the horizontal cylinder capper is said to substantially reduce the cost of capping. The cylinder rests in radius blocks at each end, and both ends are capped without moving it, making caps parallel at right angles to the axis. Feeler gages and end clamps restrict cap area and thickness, are said to insure caps of equal thickness at the thinnest point. Of aluminum and magnesium construction, the capper weighs less than 30 lb packed for shipment.—*Forney's Inc., Tester Division, P. O. Box 310, New Castle, Pa.*

ACI Concrete Primer

The 1958 edition of the ACI Concrete Primer brings developments of the past three decades into the question-and-answer format of this long-popular handbook. Expanded to 72 pages, the revised edition retains its handy pocket size. It develops in simple terms the principles governing concrete mixtures, and shows how a knowledge of these principles and of the properties of cement can be applied to the production of permanent structures in concrete.

American Concrete Institute
P.O. Box 4754, Redford Station
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Portable block splitter

Portable, hand-operated block splitter is adaptable to yard or job-site work. The manufacturer states that 11 instantly self-leveling blades on each cutting head give this 16½ in. blade span unusual versatility. Feeding table is scored for 1 in. measuring and squaring; 20 lb hand pressure creates greater than 70,000 lb breaking pressure; machine is reportedly built to withstand rigors of yard and job-site work.—E & R Manufacturing Co., 730 Pontiac St., Rochester, Ind.

Literature Available

Pertinent details on the latest equipment and products on the market are available in recently released literature. Exact titles of the booklets and catalogs are indicated in capital letters. They may be requested directly from the manufacturers listed below.

EPOCAST COATINGS—bulletin describing coatings, sealants, adhesives, and pastes made from furane resins, epoxies, and polyurethanes. Suggests new applications of plastic coatings and recommends proper coating for various uses. Illustrations show typical industrial applications. Folder includes material selection guide, chemical resistance comparison chart, and information on miscellaneous products.—Furane Plastics, Inc., 4516 Brazil St., Los Angeles 39, Calif.

PROPER USE OF CONCRETE ADMIXTURES—Twenty-page primer by R. A. Jessen explaining the use of admixture to improve quality and uniformity in concrete. Discusses comparative tests, selection of proper admixtures, concrete uniformity, quality control, and evaluating field tests. Fully describes three basic admixtures: Plastiment (retarding densifier); Sikacrete (accelerating densifier); and Sika AER (air entraining resin).—Sika Chemical Corp., Passaic, N. J.

STADIUM AND AUDITORIUM CONCRETE QUALITY AND DURABILITY (MBR-P-12)—Construction stories of 16 outstanding stadium and auditorium projects in this country and abroad cite the role played by Pozzolith in obtaining the desired handling properties of concrete during placement and in meeting requirements for concrete in the hardened state.

Among the jobs included in this 20-page study are: the Dallas Memorial Auditorium and its unique lightweight concrete roof design; the placing of the reinforced concrete dome roof of the Albuquerque Civic Auditorium; and the thin shell concrete roof design at the University of Cincinnati Armory.—The Master Builders Co., 7016 Euclid Ave., Cleveland 3, Ohio

GUNITE CONCRETE—SAND AND CEMENT PLACED BY AIR—Bulletin No. 20 has been prepared for the purpose of submitting evidence of the merits of pneumatically applied concrete through photographs of completed jobs and illustrations and descriptions of its many and varied uses. The 32-page brochure explains the process, giving general information on the physical properties, compressive strength, weight, density, and fire resistance qualities. Large photographs show various ways in which the process has been employed on structures of unusual shape and design even during inclement weather and below freezing temperatures; shows use on reservoirs, ponds, basins, canals, and channels. Contains section on design details and general specifications. —Gunite Concrete & Construction Co., 1301 Woodswether Rd., Kansas City 5, Mo.

THE PRESCON SYSTEM CUTS SCHOOL BUILDING COSTS—One of a series of folders on specific jobs uses of post-tensioned prestressed concrete. Illustrates construction features of an actual building; shows estimated costs for the 7700 sq ft area of the roof; cites economies and advantages resulting from the use of the Prescon System of prestressing concrete in school buildings.—The Prescon Corp., P. O. Box 4186, Corpus Christi, Tex.

PROPORTIONING GUIDE FOR CONCRETE MIXES CONTAINING FLY ASH—Highly informative folder explaining proper use and advantages of fly ash as an ingredient in concrete. Contains series of tables for selecting proportions for mixes containing fly ash.—Chicago Fly Ash Co., 228 N. LaSalle St., Chicago 1, Ill.; Walter N. Handy Co., Inc., 910 Custer Ave., Evanston, Ill.; McNeil Brothers, Inc., P. O. Box 4015, Bridgeport 7, Conn.

VIBRALOY WOVEN WIRE SCREENS—a 15-page comprehensive catalog covers complete line of woven wire screens. Illustrates the extensive modern wire drawing, tempering, and testing facilities of Audubon Wire Cloth Corp., as well as the company's complete line of crimping dies and tools and specially-built production equipment. Section devoted to screen definitions, wire sizes, and weights, explains how to figure wire size, mesh, and space.—Audubon Wire Cloth Corp., Allen St. and Castor Ave., Philadelphia 34, Pa.

HUMBOLDT TESTING EQUIPMENT CATALOG NO. 21-B—illustrated 48-page testing equipment catalog for cement, concrete, soils, bituminous, and petroleum products including testing sieves and shakers, balances and weights, drying ovens, and miscellaneous equipment.—Humboldt Mfg. Co., 4466 North Harlem Ave., Chicago 31, Ill.

SUBURBAN OFFICE BUILDING, FLEXICORE FACTS NO. 80—brochure illustrating a structure recently constructed in Madison, Wis., having steel frame structure with precast concrete Flexicore floors and roofs. Cross section line drawings show location of precast units in building.—Flexicore Co., Inc., 1932 E. Monument Ave., Dayton 1, Ohio

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ACI publications in large current demand

Manual of Standard Practice for Detailing Reinforced Concrete Structures (ACI 315-57)

Of large format, bound to lie flat, the manual presents recommended methods and standards for preparing drawings for the fabrication and placing of reinforcing steel in reinforced concrete structures. The previous ACI standards on detailing of buildings and detailing of highway structures have been combined into one manual. Typical engineering drawings (and, for buildings, placing drawings as well) illustrate the use of the standards. Price—$4.00; to ACI Members—$2.50.

ACI Standards—1958

Thirteen ACI Standards in one book, 356 pages, including: Building Code Requirements for Reinforced Concrete (ACI 318-56); Minimum Standard Requirements for Precast Concrete Floor and Roof Units (ACI 711-58); Evaluation of Compression Test Results (ACI 214-57); Winter Concreting (ACI 604-56); Selecting Proportions for Concrete (ACI 613-54); Measuring, Mixing and Placing (ACI 614-42); Application of Portland Cement Paints (ACI 614-49); Construction of Concrete Farm Silos (ACI 714-46); Application of Mortar by Pneumatic Pressure (ACI 805-51); Design and Construction of Reinforced Concrete Chimneys (ACI 505-54); Concrete Pavements and Concrete Bases (ACI 617-58); Design of Concrete Pavements (ACI 325-58); Test Procedure to Determine Relative Bond Value of Reinforcing Bars (ACI 208-58). Price—$4.00; to ACI Members—$2.00.

ACI Concrete Primer (1958)

The 1958 edition of the ACI Concrete Primer brings development of the past three decades into the question-and-answer format of this long-popular handbook. Expanded to 72 pages, the revised edition retains its handy pocket size. It develops in simple terms the principles governing concrete mixtures, and shows how a knowledge of these principles and of the properties of cement can be applied to the production of permanent structures in concrete. Price—$1.00; to ACI Members—$0.50.

Bibliography on Prestressed Concrete (1955)

This 102-page book lists 2100 American and foreign literature references on the subject of prestressed concrete, published 1896 through 1954. A separate section lists some 60 United States, British, German, and French patents. The 8½ x 11-in. format is also punched for easy filing in three-ring binders. Price $2.00.

Supplement to 1954 edition is available for 35 cents.

Cellular Concretes (1954)

Prepared for ACI by Rudolph C. Valore, Jr. A review is given of methods of preparation and physical properties of moist and high-pressure steam-cured cellular concretes, ranging in density from 10 to 100 lb per cu ft, as they have evolved in Europe in the past 30 years. Data are given for Swedish, Danish, Russian, German, British, and Belgian materials, and for experimental mixtures prepared at the National Bureau of Standards. 44 pages, price $1.00.

ACI Manual of Concrete Inspection (1957)

This 240-page pocket-size book, bound in durable cover, sets up what good practice requires of concrete inspectors and background information on the "why" of such good practice. This fourth edition was revised and extended by Committee 611 to include new practices evolved since the third edition of 1955. Price $3.50—to ACI Members $1.75.

Reinforced Concrete Design Handbook (1955)

An extensive revision by a reconstituted Committee 317 of the original well-known edition of 1939. Tables are revised to reflect changes in range of working stresses encountered in general practice. The original objective to reduce the design of members under combined bending and axial load to the same simple form as is used in the solution of common flexural problems is retained. Price $3.50—to ACI Members, $2.00.

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Current ACI Standards

Test Procedure to Determine Relative Bond Value of Reinforcing Bars (ACI 208-58)
16 pages: $1.00 per copy (50 cents to ACI Members)

Recommended Practice for Evaluation of Compression Test Results of Field Concrete (ACI 214-57)
20 pages in covers: 75 cents per copy (60 cents to ACI members)

Manual of Standard Practice for Detailing Reinforced Concrete Structures (ACI 315-57)
A publication of large format, bound to lie flat
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Recommended Practice for Winter Concreting (ACI 604-56)
24 pages in covers: 75 cents per copy (50 cents to ACI Members)

Recommended Practice for Selecting Proportions for Concrete (ACI 613-54)
16 pages in covers: 50 cents per copy (40 cents to ACI Members)

Recommended Practice for Measuring, Mixing and Placing Concrete (ACI 614-42)
28 pages in covers: 50 cents per copy (40 cents to ACI Members)

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